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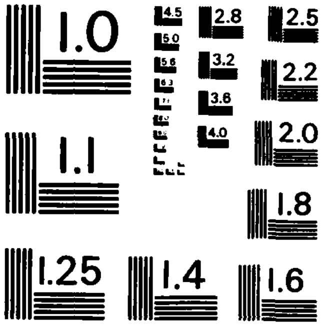

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DESIGN OF ALTERNATE LAUNCH AND RECOVERY SURFACES FOR ENVIRONMENTAL EFFECTS

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JANUARY 1982 - OCTOBER 1983

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This report documents a research effort to develop pavement design requirements for Alternate Launch and Recovery Surfaces (ALRS) that will minimize the cost and reduce deterioration due to environmental (i.e. climate and aging) factors. A minimum asphalt surface thickness of 2 inches was established through traffic testing of three test items. Seven environmentally aged pavements in nontraffic areas (shoulders, overruns, etc.) were trafficked to failure. Results from these tests indicated that the (continued)

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20. ABSTRACT (Continued)

base course strength was below the design strength and therefore pavements failed before design estimates.

A laboratory aging test was developed to simulate environmentally-aging asphalt mixtures over a short period of time to produce asphalt binder properties similar to those experienced in the field after aging from 10-15 years. Based on these tests, specification requirements were recommended to minimize asphalt hardening.

A frost-design procedure was recommended for ALRS pavements based on reduced subgrade strength. Recommended design, construction, and maintenance procedures were presented.

PREFACE

This report was prepared by the Geotechnical Laboratory, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi 39180 under Military Interdepartmental Purchase Requests N-82-34 and N-83-5, for the Air Force Engineering and Services Center, Tyndall Air Force Base, Florida. The subcontractor for the section dealing with design for frost conditions was the U. S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755.

This report summarizes work done between January 1982 and October 1983. The AFESC/RDCR project officers were Major Thomas E. Bretz, Captain Henry F. Kelly and Captain John D. Wilson.

The report documents design requirements for a thin asphalt surfacing to minimize the effects of environmental aging.

This report has been reviewed by the Public Affairs Office and is releasable to the National Technical Information Service (NTIS). At NTIS it will be available to the general public including foreign nationals.

This technical report has been reviewed and is approved for publication.

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SECTION I

INTRODUCTION

A. BACKGROUND

The US Air Force needs to construct and maintain Alternate Launch and Recovery Surfaces (ALRS) in Europe and the Republic of Korea. With the widespread construction of hardened aircraft shelters which greatly reduces the vulnerability of aircraft on the ground, the high-quality existing airfield pavements offer an excellent target for the enemy to effectively neutralize US air power. To counteract this threat, ALRS can be constructed in certain areas to effectively reduce the probability that all landing-takeoff areas would be destroyed. The ALRS must (1) be relatively inexpensive in comparison to permanent pavements, (2) support the imposed loads, (3) be easily maintained, and (4) provide an adequate surface for a limited number of sorties of the design aircraft.

Alternatives for ALRS being studied presently include: (1) stabilized soil base, (2) crushed stone base, and (3) reinforced soil base, all with some type of wearing surface. The work reported herein will concentrate on asphalt-surfaced pavements with consideration of both thin asphaltic concrete and double bituminous surface treatment over a granular base.

The ALRS surfaces will only be used in contingency situations when the runways are damaged, but are to be designed for a 20-year life. The pavements are to be located where there are 300-1000 freezing degree-days, 25-30 inches of rainfall, and 14-36 inches of snowfall per year (Reference 1). These environmental conditions will deteriorate the strength properties of the pavements through thermal cracking, freeze-thaw, and water infiltration through cracks.

The ALRS have been proposed for construction at a specific number of air bases. The subgrades at these bases are predominantly fine-grained materials with California Bearing Ratios (CBRs) ranging from 2-16. The pavements are to be designed for 150 passes of an F-4 aircraft. The F-4 has a single-wheel main gear with a maximum gear load of 27,000 pounds and a 111-square inch contact area.

B. PURPOSE

The purpose of this research effort was to: (1) define the minimum pavement structural design requirements for the ALRS, (2) evaluate long-term deterioration from environmental factors, (3) develop an asphaltic concrete mix design to minimize the environmental deterioration of the ALRS, and (4) demonstrate the performance of the F-4 aircraft on the minimum design ALRS.

These objectives were achieved through: (1) a review and modification of existing design criteria to provide the minimum design requirements, (2) an assessment of existing available information on environmental deterioration of pavements, (3) a field-performance evaluation of environmentally deteriorated pavements using simulated F-4 traffic (load cart), and (4) actual

F-4 aircraft demonstrations on selected minimum design pavements. Data from pavements in Europe and the Republic of Korea that have been subjected to the particular climatic and aging environments of interest were examined in terms of surface pavement condition to study the long-term environmental effects. Typical pavements at military bases in the U. S. were load-tested to determine long-term deterioration in load-support capacity.

C. APPROACH

The Corps of Engineers has thickness design criteria for low-pass levels such as those anticipated on the ALRS. These requirements were considered to be conservative. An evaluation of these criteria for the loading conditions anticipated on the ALRS was made by constructing three test items of different thicknesses and compositions at the Waterways Experiment Station (WES) and trafficking these items with the F-4 loading.

Current design criteria for the F-4 require 3 inches of asphalt surfacing as a minimum thickness for permanent pavements under continuous use. For limited use, this requirement could be reduced to a thinner layer and perhaps a surface treatment would suffice. Tests were conducted to determine the minimum thickness of asphalt needed to support the F-4 for the ALRS. These included straight-line traffic, turning movements, and locked-wheel skids.

Material requirements for base and subbase courses have been developed in the past for use in permanent airfield pavements. From these initial material requirements it is assumed that minimum strength properties will be retained over the pavement's life. These strength properties were evaluated for pavements that were environmentally aged for 10 to 30 years.

Since the ALRS pavements will not be subjected to the traffic except during contingency situations, environmental deterioration is of paramount importance in the design. Temperature and moisture are fundamental variables in all problems of airport pavement construction, design, behavior, and performance. Climatic parameters important to the pavement design were reviewed.

Specifically, the objectives of this phase of the investigation were to: (1) review the current state of the art in the design of pavements for environmental conditions; (2) identify critical types of distress caused by environmental conditions important to the design of ALRS, with particular emphasis on those distress types that would hinder operations of the F-4 aircraft; and (3) identify critical pavement design parameters affected by environmental conditions and conduct a sensitivity analysis on these parameters using available data so that design limits may be established.

Since the ALRS will be located in areas with design freezing indexes of up to 1000 freezing degree-days, a frost-effects design procedure was developed to minimize the life-cycle cost over a 20-year period. The minimum base thickness design and use of geotextiles (membrane-encapsulated soil layers) were evaluated for cost effectiveness in the ALRS design.

To evaluate the long-term deterioration of nonuse pavements, field tests using prototype loads were conducted on representative pavements. Some

pavements at various military airfields receive little or no traffic but have been subjected to long-term environmental effects. These pavements are found at the outside portion of runways, taxiways, and aprons. Seven representative asphalt-surfaced pavements were selected, covering a range of environmental conditions and falling within the minimum design considerations. Load tests were conducted on these pavements, using a moving-wheel load cart that simulates the F-4 aircraft wheel loads. Results from these tests were compared to the traffic tests on the new minimum design pavements described earlier, and to the design predictions for the pavements as if they were new construction.

The field tests consisted of characterization of each test area through test pit measurements to determine material types, thicknesses, CBR, and moisture-density. Available records as to original design/construction were obtained. Environmental data such as temperature, freeze-thaw, and rainfall were gathered for the life of the pavement. Pavement condition of the entire area was evaluated in terms of the Pavement Condition Index (PCI).

SECTION II

REVIEW OF PAST TRAFFIC TESTS

The initial phase of the Alternate Launch and Recovery Surfaces (ALRS) project was to conduct a literature survey to determine those instances in which bituminous surface treatments or thin asphaltic concrete (AC) surface layers had been subjected to traffic by high-pressure tires. The majority of the information available was related to roads and streets trafficked with relatively light loads and low tire pressures (cars, trucks, etc.). However, data were available from tests where large loads with high tire pressures were applied to surface treatments and thin AC layers. The tire pressures ranged from a low of 100 psi to a high of 300 psi. The loadings ranged from a low 8000-pound single-wheel load to a high 150,000-pound dual-wheel load. The ALRS project is concerned with single-wheel loads of 27,000 pounds and a tire pressure of 265 psi, so that the available data did bracket the proposed design loading. A summary of the reports containing applicable data is presented in the following paragraphs.

Reference 2 presents the results of traffic applied to a flexible pavement test section consisting of a well-graded crushed limestone base course constructed on a weak clay (CH) subgrade having a 6 CBR. The test section consisted of three items containing 5 inches, 8 inches, and 11 inches of base course material surfaced with a bituminous surface treatment. Base course CBRs varied from 85 in Section I to 153 in Section III at the start of traffic but were reduced to a 60 CBR prior to the completion of 100 applications of traffic.

Basic data relative to the test cart used to apply traffic to the test sections are shown below:

Single-Wheel Load, lbs.	Tire Size	Inflation Pressure, psi	Contact Area sq. in.
10,000	34-9.9	100	91
25,000	56-16	100	232
50,000	25-28	100	479

In all wheel load tests, the surface treatment cracked significantly as deformations occurred in the underlying base course. The cracking and breaking up of the surface treatment indicated that it would not be adequate for a long-term ALRS surfacing.

Reference 3 presents data to validate thickness and compaction requirements for flexible pavements subjected to channelized traffic. A test section consisting of four test items was constructed for this study. The various materials, thicknesses and CBR values are tabulated on the next page.

Item 1

1 in. Double Bituminous Surface Treatment (DBST)
12 in. Crushed Limestone (CBR-80)
29 in. Sand Gravel (CBR-45)
24 in. Clay (CH) Subgrade (CBR-10)

Item 2

1 in. DBST
12 in. Crushed Limestone (CBR-80)
22 in. Sand Gravel (CBR-45)
24 in. Clay (CH) Subgrade (CBR-10)

Item 3

1 in. DBST
12 in. Crushed Limestone (CBR-80)
15 in. Sand Gravel (CBR-45)
24 in. Clay (CH) Subgrade (CBR-10)

Item 4

1 in. DBST
14 in. Crushed Limestone (CBR-80)
96 in. Sand Subgrade (CBR-25)
9 in. Limestone

Original plans were to apply 30,000 coverages of traffic to the test section with a 100,000-pound twin-wheel assembly load with tires inflated to 200 psi and a contact area of 267 square inches, thus simulating the channelized traffic of a fully loaded B-47 aircraft. However, as traffic-testing progressed, two modifications were made to the test program. The first modification involved an increase in load to 150,000 pounds; tire pressure was increased to 300 psi to maintain a constant contact area 267 square inches; and the second modification involved a reduction of 3 inches in the test section's total thickness for Items 1, 2, and 3. A brief summary of the three phases of traffic testing is as follows:

> Phase 1 consisted of 9142 coverages with 100,000-pound twin-wheel assembly load, 200-psi tire pressure; no failures encountered....
>
> Phase 2 increased twin-wheel assembly load to 150,000-pound, 300-psi. 4000 coverages, no visual distress in Items 1 and 2. However, some cracking developed in Items 3 and 4. Due to frequent tire failures thought to be caused by protruding aggregate, a 1/2-inch sand-asphalt layer was placed over the entire test section. Noticeable settlement occurred in Item 4....

Phase 3 The sand-asphalt layer and about 3 inches of base were removed from Items 1, 2, and 3 and a new sand-asphalt surface was applied. Item 4 was not changed. 7186 coverages were applied. Items 1 and 2 performed satisfactorily; Items 3 and 4 were considered borderline....

These tests demonstrated the ability of surface treatment to resist damage from high tire pressure and heavy loads when placed on a high-quality base course.

Reference 4 presents results of tests to determine the strength of base courses required directly under an asphaltic concrete wearing surface. A test section was constructed having three traffic lanes and was trafficked using dual wheels loaded to 60,000 pounds, 91,000 pounds, and 120,000 pounds. The tire pressures were 100, 170, and 240 psi, respectively. The contact area for all loadings was maintained as close to 267 square inches on each tire as possible. The test section contained four different 22-inch-thick base course materials surfaced with a DBST. The base course materials were a crushed limestone, clayey gravel, clayey sand, and a lean clay. All test items performed satisfactorily under 2,000 coverages of the 60,000-pound, 100 psi load, with no failures developing. After application of 2,000 coverages to the test section with the 91,000-pound, 170 psi loading, there was some shoving of the DBST in the limestone test item. The clay gravel test item exhibited some distress at 1,800 coverages due to rutting in the base course. The surface treatment on the clayey sand base was cracking and breaking up at 800 coverages, due primarily to movement in the top 2 inches of the base course. The DBST on the lean clay base performed better than all other tests. At 2,000 coverages, there was no evidence of distress.

The 120,000-pound, 240 psi loading produced failures in all test items. In the limestone test item, there was some shoving at 2,000 coverages due to movement in both base course and surface treatment. The clayey gravel section failed very early (76 coverages) as did the clayey sand section (52 coverages) due to extensive rutting and shear deformation. The lean clay section sustained 2,000 coverages with only minor cracking and rutting of the base course.

The primary purpose of the work described in Reference 5 was to investigate and determine techniques for construction of a waterproof fine-grained soil base course by encasing the soil layer in a protective membrane envelope and to determine the effects of aircraft traffic on such a base course. Traffic was applied by using a single-wheel load of 25,000 pounds. The load cart was equipped with a 30 by 11.5, 24-ply rating tire inflated to 250 psi. The tire had a contact area of 111 square inches.

After a cumulative total of 502 coverages a double bituminous surface treatment (DBST) was placed on a membrane above a highly compacted lean clay base (Items 1 and 2). Traffic was continued to 580 coverages. After completion of traffic, the test items were in excellent condition with no cracking or breaking up of the DBST. These 78 coverages showed the ability of a DBST to withstand F-4 traffic (25,000 pounds, 250 psi) applied as rolling wheel loads. Skid tests were conducted after traffic and the DBST could not withstand the effects of these tests as it broke up.

Reference 6 presents results from traffic tests using an 8,000-pound, single-wheel load. The high-pressure tire (24 by 5.5 inches) was inflated to 240 psi and had a contact area of 37 square inches. Two test items noted were constructed using 9 inches of crushed limestone base course over a subgrade having a CBR of 15. The surface was 1 1/2-inch thick AC. Only minor rutting and cracking of the AC was reported after 1,000 coverages. Although these two items performed well, they were subjected to a load much less than that required for the ALRS.

A literature survey was conducted to determine the performance of surface treatments and/or thin layers of AC when subjected to high-pressure tires. Several reports were reviewed (References 2-6) and these basically showed that the surface treatments or thin asphalt surface layers were capable of sustaining rolling wheel traffic for the number of passes required for the ALRS. Although failures did occur in most tests, they were due to some cause or something other than the surface treatment or AC layer itself, and were generally at a pass level in excess of that required for ALRS. Skid tests and turning wheels were detrimental to surface treatments and caused quick failures.

SECTION III

TRAFFIC TEST SECTION

A. DESIGN

1. General

Current design criteria (Reference 7) for the F-4 require 3-4 inches of asphalt surfacing, depending on the base course strength. These criteria are considered conservative for ALRS-type pavements. This report evaluates the current wearing course design criteria to reduce conservatism and meet the requirements to withstand 150 passes of an F-4.

A traffic test section was constructed at the WES. The subgrade of the test section was constructed for a 6 CBR $\pm$ 1. The strength was selected from typical values for soils at U. S. airbases in the Federal Republic of Germany given in Reference 1. Using the flexible pavement design procedure (Reference 7), a CBR of 5, gross weight of 60 kips, and 150 aircraft passes, yields a required total pavement thickness of 12 inches above the subgrade. Three wearing surfaces, a double-bituminous surface treatment (DBST), a 1-inch AC surface, and a 2-inch AC surface were evaluated in this test section. Three test items meeting the 12-inch total thickness over a 5 CBR subgrade are shown in Figure B-1 and described as follows:

Item Number	Construction
1	2 in. Asphaltic Concrete 10 in. Crushed Stone Base
2	1 in. Asphaltic Concrete 11 in. Crushed Stone Base
3	1 in. Double-Bituminous Surface Treatment 11 in. Crushed Stone Base

2. Subgrade

For a subgrade soil to simulate the approximate design strength expected for ALRS type pavement, a material was selected that will maintain nearly the same properties from construction until after traffic. The material commonly called "Vicksburg Buckshot Clay" is classified as a CH soil, according to the Unified Soil Classification System (USCS). Classification data for this material are shown in Figure B-2. Laboratory compaction and CBR data for the as-molded and soaked conditions are shown in Figures B-3 and B-4. These data indicate an unsoaked CBR of about 20-25 for a water content of about 22 percent and a dry density range of 95 to 105 pounds per cubic foot (PCF). A soaked CBR of 5-7 can be obtained with a water content of 25-29 and a dry density of 90 to 94 pcf.

3. Base Course

The material used for the base course of the ALRS test section was a crushed limestone. Classification data are shown in Figure B-2. Laboratory compaction and CBR data for the as-molded and soaked conditions are shown in Figures B-5 and B-6. These data indicate that if a dry density of above 135 pcf is obtained, the CBR should be above 100.

4. Surface Treatment

A double-bituminous surface treatment (DBST) was selected for the surface treatment design. A CRS-2 emulsified asphalt was selected as the binder material. Gradations for the two aggregate courses are shown in Figure B-7.

5. Asphaltic Concrete

An AC surface mix was designed in accordance with the Marshall design method given in MIL-STD-620. The laboratory mix design was obtained by combining three aggregates and AC-20 asphalt cement to meet the specification requirements. Gradation for the aggregates, the approved mix, specification limits, and results from Marshall stability tests are shown in Table A-1. Aggregates selected were a crushed limestone of coarse and fine gradations, and a local concrete sand. The AC mix was manufactured by a local contractor.

The aggregate stockpiles available for use in construction of the test section at WES could not be blended to precisely meet the specification requirements and the job mix formula was slightly outside the specified limits for gradation on the 1/2-inch, 3/8-inch, No. 4, No. 50 and No. 200 sieves. The aggregate gradation did not significantly deviate from the specifications on critical sieves; therefore, the mix was satisfactory. All mixture properties for the job mix formula were within the specification requirements for airfields.

6. Instrumentation

All three items were instrumented with linear variable differential transformer (LVDT) displacement transducers to measure vertical surface deflections. The LVDT produced dc output voltages directly proportional to the movement of the sensing unit. The transducer consisted of a main body, which housed the sensing coil and its associated electronics, and a movable core through the center of the sensing coil to transfer the mechanical movement of the core to a change in an electrical signal in the coil. The LVDT transducers were mounted on reference rods that extended to references flanges located approximately 6 feet below the bottom of the test bed. The reference rods were cased with 2 inch PVC pipe attached to the gage housing with flexible hose (Photo C-1). The gages were placed outside the traffic lane in the center of each item and 7 feet from the center of the lane for static testing with the F-4 load cart.

B. CONSTRUCTION

1. General

Excavation, construction of the structural layers, and final paving phases of the test section were performed during March, April, and May 1982. First, excavation to a depth of 32 inches below the existing grade was accomplished. This excavated area was 20 feet wide and 150 feet long.

2. Subgrade

The material at the base of the excavation on which the subgrade material was to be placed was classified as a CL with a CBR of about 30. Prior to placement, the CH clay was processed to the desired water content. The clay was then placed and compacted in four lifts of 6 inches each over all three test items. The subgrade was compacted with a 50-kip rubber-tired roller (Photo C-2). Compaction and CBR data for all items before and after traffic are shown in Table A-2.

3. Limestone Base

The limestone base course was placed in two lifts to achieve a 10-inch thickness in Item 1 and an 11-inch thickness in Items 2 and 3. The limestone was compacted with a vibratory steel-wheeled roller, a 50-ton roller (Photo C-3), and a rubber-tired roller to achieve the desired density. Compaction and CBR data for all items before and after traffic was applied are shown in Table A-2.

4. Asphalt Concrete

Prior to placing the DBST and the AC surfaces, the base course was primed with an EA-1 penetrating asphalt emulsion. This material is an SS-1 emulsion with kerosene added. The AC for the wearing course was mixed by a local contractor according to the design specifications and hauled to the test section site. It was placed by a Barber-Greene asphalt finisher in 10-foot wide lanes along the test section. Placement temperature was about 350°F. Placement was accomplished in one lift, with the final thickness being 2 inches on Item 1 and 1 inch on Item 2. Shortly after placement, the mixture was compacted by breakdown rolling with 1-3 coverages of a vibratory steel-wheeled roller, 32 coverages of a 50-kip rubber-tired roller, and 4 more coverages of the vibratory steel-wheeled roller (Photo C-4).

Samples of the aggregate were collected from the hot bins. The gradation is shown in Table A-3. Core samples were taken from the surface in place. Extractions were made to determine the asphalt content and gradation of the aggregate. Samples were recompacted and tested for Marshall stability. Results are shown in Table A-3.

A review of Table A-3 indicates that the gradation of the aggregate in the mixture produced for the test section was essentially equal to the job mix formula. The test results did indicate however that the asphalt content (4 percent) was lower than the optimum asphalt content (5 percent). Part

of this difference in test results could have been caused by the variability of the asphalt extraction test; however, the mix properties do indicate that the asphalt content was low. The low asphalt content would affect the durability of the asphalt mixture, but the effect of the performance under accelerated traffic would be minimum. This difference in asphalt content from mix design and construction does highlight the need for close quality control during construction of prototype pavements. In the construction of small test sections, the small amount of material being placed makes it impossible to identify problems and make corrections before completion of construction. However, on a full-scale job, these deficiencies can be identified and corrected during construction.

Densities were also calculated from the cores taken in Item 1, the 2-inch AC surface and Item 2, the 1-inch AC surface. The density in Item 1 was 143.6 pcf or 98 percent of laboratory density (see Table A-1). Densities in Item 2 ranged from 90.3 to 93.0 percent of laboratory density. The edges of the samples cored in Item 2 were ragged. This may have caused the lower density values.

5. Surface Treatment

Construction of the DBST consisted of placing a CRS-2 asphalt emulsion on the primed base course at a rate of 0.3 gallons per square yard. The first course of crushed stone was applied and seated with a 50-kip rubber-tired roller. Loose material was removed by brooming from the section and the placement process was repeated.

6. Instrumentation

The LVDT gages were placed in 3-inch core holes which were made in each item approximately 7 feet from the centerline of the section (Photo C-5). The LVDT gages, the reference rods, and the flexible casing were placed in the core holes and epoxyed into place and the surface plate was epoxyed into the surface of each item (Photo C-6).

C. TRAFFIC

1. General

Traffic tests were performed on the test section from 24 May to 3 June 1982. The test cart, traffic patterns, failure criteria and performance of the test section during traffic are described in the following paragraphs.

2. Test Cart

The F-4 load cart, shown in Photo C-7, was used to traffic test all three test items. The cart was loaded to 27,000 pounds and used a 30 x 11.5-14.5, 24-ply rating tire inflated to 265 psi, resulting in a tire contact area of 111 square inches.

3. Traffic patterns

Each of the test items was trafficked with both a distributed and a channelized pattern. The distributed traffic pattern is shown in Figure B-8. To apply the traffic, the test cart was driven backward and forward along the same path, then shifted laterally the distance equal to one tire width (10 inches) and the process repeated. The interior 40 inches received 100 percent of the maximum number of passes in any wheel path and the exterior portions of the lane received 67 and 33 percent. This pattern corresponds to the 70-inch wander pattern normally used in the design of taxiways and runway ends. The channelized traffic pattern consisted of repeated passes in only one lane.

In addition to the traffic testing, locked-wheel skid tests were performed. For the skid tests, the load cart wheel was locked and the test cart towed a minimum of 5 feet. The skid tests were performed in a channelized traffic pattern.

4. Failure Criteria

The failure criteria selected for ALRS pavements consist of the following:

a. Base course aggregate exposure sufficient to pose a foreign object damage (FOD) potential;

b. AC presents FOD potential;

c. A rut depth in excess of 3 inches;

d. Other conditions, as determined by the project engineer, that cause the pavement to be nonserviceable.

Whenever one of these failure criteria was reached on a given item under a given loading, the traffic was discontinued and data were recorded.

D. TEST RESULTS

1. Behavior of Pavement Under Traffic

Behavioral observations of the test items were recorded throughout the traffic test period. These observations were supplemented by photographs. After failure, thorough examinations of the failed areas were made, including CBR tests, water content determinations, and dry densities of the different pavement structural layers to determine the effects of the various traffic patterns on these parameters. The behavior of each item under traffic is given below.

Item 1 consisted of 2 inches of AC over 10 inches of crushed limestone base over 24 inches of heavy clay subgrade. Under distributed traffic, failure occurred at 338 passes with the observance of a 3 3/4-inch rut depth (Photo C-8). Channelized traffic caused failure after

54 passes when a 3 3/16-inch rut depth was measured (Photo C-9). The skid test failed Item 1 after four skids in the same location with a rut depths exceeding 3 inches (Photo C-10).

The pavement structure in Item 2 consisted of 1 inch AC over 11 inches of crushed limestone base. The distributed traffic pattern caused failure after 150 passes with a 3-inch rut depth (Photo C-11) while failure under channelized traffic occurred at 41 passes with a 3-inch rut depth (Photo C-12). Two passes of the skid test failed Item 2 with a 3-inch rut depth (Photo C-13).

Item 3 was composed of a DBST over 11 inches of crushed limestone base. Distributed traffic caused failure after 48 passes with a 3-inch rut depth (Photo C-14). Channelized traffic failed the item after 29 passes with a 3 5/16-inch rut depth (Photo C-15). One locked-wheel skid test created a 3-inch rut in the pavement (Photo C-16).

Results of traffic and skid tests on the ALRS test section are presented in Table A-4. Typical cross sections and profiles are shown in Figures B-9 to B-14. From the construction and before-traffic cross-section data, the average layer thicknesses were determined. These values and the standard deviations are given in Table A-5. Final thicknesses were less than the design except for the AC thickness in Item 2. From these data and the CBR data given in Table A-2, the CBR design procedure was used to predict the passes to a 1-inch rut depth (see Table A-6). Excellent agreement was obtained for all items.

Skid test results indicate that the DBST would be removed with initial braking. Failure occurred in the AC surfaced items after four skids and two skids. Failure was due to rutting of the pavement under the locked wheel and would not be expected with the use of antiskid devices on the F-4 aircraft.

Deflection basins were measured in each item for the F-4 loading by placing the load cart 4 feet from the LVDT gages, allowing each gage to stabilize for 2 minutes and reading the gage. The load cart was then moved to 3 feet from the gage and the procedure repeated. This method was also used for 2- and 1-foot distances. The load cart was not positioned over the gage because the gage would overrange. Results of these tests are shown in Figure B-15. The AC surface lifted for distances of 3 and 4 feet but the DBST did not.

E. SUMMARY

Three test pavements, designed for ALRS loading conditions, were constructed and tested under accelerated traffic. Using a 3-inch rut depth failure criteria, both the 1- and 2-inch AC surfaced items met or exceeded the required 150 passes of distributed traffic. The DBST did not adequately support the required traffic. From these results the minimum thickness requirements for ALRS design can be reduced from 3 inches to 2 inches for the F-4 aircraft with a gross aircraft load of 60,000 pounds and a tire pressure of 265 psi. Since results for the 1-inch AC thickness are marginal, a 2-inch AC minimum thickness is recommended.

SECTION IV

TRAFFIC TESTS ON ENVIRONMENTALLY AGED PAVEMENTS

A. TEST CONDITIONS AND PROCEDURES

1. General

The design freezing index was used as the basis for selection of test pavements that had been environmentally aged under conditions similar to those in Germany and Korea where ALRS pavements are to be built. Sites evaluated for testing included Fort Devens, Massachusetts; Seneca Army Depot, New York; Wright-Patterson AFB, Ohio; and Whiteman AFB, Missouri. Wright-Patterson AFB and Whiteman AFB were selected, based on the design freezing index and because more pavement areas were available in fewer locations minimizing transportation costs. The design freezing index for Wright-Patterson and Whiteman AFBs were 892 and 686 freezing degree-days, respectively.

The areas selected for traffic tests, at both Wright-Patterson and Whiteman AFBs, consisted of taxiway shoulder pavement, apron pavement and a parking pad for fire equipment. All of the traffic test features, except one, were constructed of AC. One feature, a runway overrun, with a DBST was also trafficked. A layout of the airfield pavement and the location of the test features are shown in Figures B-16 and B-17.

Because of the non-traffic-type pavements selected, it was difficult to locate detailed information pertaining to construction dates and maintenance records. The consensus is that nontraffic pavements are constructed within the same period as the active pavement feature. A list of pertinent data including construction and maintenance dates is shown in Table A-7. The pavements ranged from 9 to 30 years old at the time of testing.

Testing was done at the two airfields mentioned above during July and August 1982. Traffic was applied to the test features with a single-wheel load test cart. The test cart, turning and locked-wheel skid tests, application of traffic and failure criteria are discussed in the following paragraphs.

2. Test Cart

A specially designed single-wheel test cart, loaded to 27,000 pounds, was used during the traffic tests to simulate the loading conditions of an F-4 aircraft. The cart was equipped with an outrigger wheel to prevent overturning and was powered by the front half of a four-wheel-drive truck. The load wheel had a 30.00 by 11.5-14.5, 24-ply tire inflated to 265 psi, which produced a measured elliptical tire print 10 by 13.5 inches (111-square inch contact area) and an average contact pressure of 254 psi. This load cart is shown in Photo C-7.

3. Turning and Locked-Wheel Skid Tests

The turning test of the load cart was made at slow speed and at the shortest turning radius of the test cart. At least 75 turning repetitions (or until failure) were applied to each feature.

The locked-wheel skid test was also conducted on each feature. The F-4 load cart wheel was locked and the test vehicle was towed for a minimum of 5 feet. Pavement surface condition was monitored and rut depth measurements were recorded during the skid tests.

4. Application of Traffic

The test traffic was applied by driving the test cart forward and backward in the same path for the length of the traffic lane, then shifted laterally a distance equal to one tire print width (10 inches) and the process repeated. The interior 40 inches received 100 percent of the maximum number of passes in any wheel path and the exterior portion of the lane received 67 and 33 percent. This pattern corresponds to a 70-inch wander which is normally used in the designs of taxiways and runway ends. It is assumed that 75 percent of the traffic does occur in this 70 inches. Assuming a normal distribution (as shown in Figure B-8), the standard deviation would be 30.43 inches.

5. Failure Criteria

The following guidelines were used to determine failure of the test pavements:

a. Base course aggregate is exposed sufficiently to pose a foreign object damage (FOD) potential.

b. Asphalt concrete debris poses a FOD potential.

c. Rutting of pavement is in excess of 3 inches.

d. Other conditions, as determined by the project engineer, causes pavement to be nonserviceable.

B. BEHAVIOR OF PAVEMENT UNDER TRAFFIC

1. General

Visual observations on the behavior of the test pavements were recorded throughout the trafficking of the seven test features and supplemented by photographs. Level readings were taken on the test pavements before and during traffic to depict any deformation that was occurring. After completion of testing on each feature, a thorough investigation was conducted. Test trenches were excavated across the traffic lane and profiles of the pavement structure were noted. Photo C-17 shows a test trench after the bituminous surface had been removed. Photo C-18 shows a test trench with both the surface pavement and the base course material removed. CBRs and

other pertinent data were obtained at the surface of the base course and at the top of the subgrade wherever possible. Samples of the different materials from the test trench were taken for laboratory testing. Classification data of the soil materials are shown in Figures B-18 and B-19.

2. WP-1 (Fire Equipment Parking Pad)

A general view of the traffic area before traffic is shown in Photo C-19. As traffic was started, rutting developed rapidly and after 44 passes the pavement was considered failed (Photo C-20). At one location during traffic the test cart became stuck in a 6-inch rut and had to be towed from the area.

Photo C-21 shows a 2-inch rut depth in the pavement after 10 passes during the turning test. However, on the 11th pass, the load wheel became stuck in the pavement and had to be towed from the test area (Photo C-22). Because of the very poor condition of the pavement surface and the potential damage aspect to the load cart, traffic testing, including the planned locked-wheel skid test, was abandoned.

3. WP-2 (Shoulder Pavement, Taxiway 17)

Photo C-23 shows the test pavement after 48 passes. As traffic was continued, attention was focused on the condition of the pavement surface. At the end of 136 passes some longitudinal and alligator cracking was noted. Photo C-24 shows some slight rutting and cracking after 240 passes. The general pavement condition after 481 passes is shown in Photo C-25. Traffic was resumed and continued until failure occurred at 643 passes. Rut depth measured at the time of failure was in excess of 3 inches. Photos C-26 and C-27 depict the failed condition of the pavement feature. Changes in the surface condition of the pavement during the F-4 load cart turning-test were closely monitored. The required 75 turns were successfully applied to the pavement feature and the recorded rut depth measurements are tabulated below:

No. of Turns	Rut Depth-Inches
37	1/2
53	1/2
75	5/8

After six skid tests on this feature only a 1 1/4-inch rut depth was measured (Photo C-28). Further skid testing had to be cancelled because of severe tire damage to the load wheel.

4. WP-3 (Parking Apron D)

Photo C-29 shows the overall view of the test area prior to trafficking the F-4 load cart. Longitudinal cracking and rutting of the pavement surface were observed after 12 passes. Data was collected after 48 passes of the test traffic. Straightedge measurements were taken at three locations within the traffic area, and the average rut depth recorded was 1 1/4 inches. Traffic was resumed and after 10 more passes, measurements showed the rut depth had

increased to 1 3/4 inches. After a total of 90 passes the pavement was considered failed due to extensive surface cracking and a rut depth in excess of 3 inches as shown in Photo C-30.

Both the turning and skid tests were applied to the WP-3 test feature. Failure occurred after 28 and 6 passes, respectively. The following tabulation depicts the numbers of passes versus rut depth-inches for both the turning and skid tests:

Turn Test		Skid Test	
Passes	Rut Depth, Inches	Passes	Rut Depth, Inches
11	1	2	1/2
19	2	4	2 1/2
25	2 5/8	6	10
28	2 7/8		

Photos C-31 and C-32 show the failed conditions.

5. WP-4 (Shoulder Pavement, Taxiway 5)

The pavement surface of feature WP-4 was considered very good to excellent with no distress observed before traffic. After 48 passes of the F-4 load cart, only a small amount of low severity alligator cracking was noted. Photo C-33 depicts the very good condition of the overall traffic area after 48 passes. A condition survey was taken after 119 passes. The pavement distresses noted during the condition survey were low severity longitudinal and alligator cracking. The condition survey rating for the test area was good. Rut depths recorded after 136 and 143 passes showed rut depths 1 7/8 and 2 1/4 inches, respectively. The pavement feature was considered as failed after 162 passes with a measured rut depth in excess of 3 inches.

A summary of the turn and skid test results is presented below:

Turn Test		Skid Test	
Passes	Rut Depth, Inches	Passes	Rut Depth, Inches
37	< 1/8	17	3/4
55	3/4	23	1 1/4
65	7/8	30	2
75	1 1/4	35	> 3

Photo C-34 shows the condition of the pavement surface after completion of 75 turns. Photos C-35 and C-36 depict the skid test area prior to traffic and after 35 passes.

6. W-1 (Runway Overrun)

The double bituminous surface treatment (DBST) of this test feature was the only surface of this type construction tested during the field studies.

Preliminary data were collected before traffic and the surface of the overrun was considered in very good condition.

After the first few passes had been applied, some longitudinal cracking was observed. However, 280 passes had been applied when the feature was considered as failed due to rutting. During these 280 passes, traffic was stopped several times to collect data and record rut depth measurements. Observations revealed that between 75 and 100 passes the DBST was starting to break up (Photo C-37). Rut depth measurements taken after 100 passes averaged approximately 1 3/4 inches. After 126 passes, a 2-inch rut depth was evident for the entire length of the traffic area. The depth of the ruts continued to increase. A depth of 2 1/2 inches was recorded after 220 passes and after 250 passes the average rut depth measured 2 3/4 inches. Photo C-38 shows the failed test feature and a rut depth in excess of 3 inches after 280 passes.

During the turning tests some cracking of the pavement along the edge of the tire print was observed. Both low and medium severity alligator cracking was noted after 75 turns had been applied to the traffic area. The turning tests on the DBST feature were considered successful. The area after the 75 turns is shown in Photo C-39.

The DBST failed after only one skid of the test load cart. Photo C-40 shows the 4 1/2-inch rut depth that was measured.

7. W-2 (Shoulder Pavement, Taxiway 9B)

A condition survey, taken on this feature before the start of traffic, noted 22 feet of longitudinal and transverse cracking. One hundred and fifty square feet of medium severity alligator cracking was also recorded. The results of the survey rated the test feature as poor.

After 50 passes, traffic was halted for observations and measurements (Photo C-41). Traffic was continued to 100 passes for measurements and photographs. At this pass level, straightedge measurements indicated an average rut depth of 2 3/8 inches (Photo C-42). Again the load cart began applying traffic to the area; after 132 passes, traffic was halted and a rut depth measurement of 3 3/4 inches was recorded. Photo C-43 shows the failed traffic area. Photo C-44 is a closeup view of the failed area showing the loose material that had spalled from the cracks.

Photo C-45 shows a general view of the condition of the pavement surface before trafficking the turn tests. After 11 turns, traffic was stopped to examine the pavement and take rut depth measurements. Photo C-46 shows the 1 1/4-inch rut depth after the 11th pass. Traffic was resumed but after a total of 27 passes and a rut depth of 3 3/8 inches, traffic was stopped and the feature was considered as failed (Photo C-47).

The locked-wheel skid test was terminated after three skids due to tire damage of the load wheel. Rut depth measurements for the three skids are shown on the next page:

No. of Skids	Rut Depth, Inches
1	3/4
2	1 5/8
3	2 1/2

Photo C-48 depicts the condition of the pavement surface and the rut depth after three skids.

8. W-3 (Alert Apron)

Photo C-49 is a general view showing part of the Alert Apron from which test feature W-3 was selected for trafficking the F-4 load cart. Traffic was stopped after 50 passes because of the significant amount of pavement distresses observed. A condition survey was performed, and the following percentages of pavement distresses were recorded: 40 percent of the test area contained low severity alligator cracking; 30 percent showed medium severity alligator cracking; 10 percent had high severity rutting; and 2 percent had high severity longitudinal cracking. A rut depth of 2 1/4 inches was measured after 50 passes (Photo C-50). After all data had been taken, traffic was continued to 86 passes at which time a rut depth of 3 1/2 inches caused the feature to be considered as failed (Photo C-51).

Turning tests were applied to the test feature and after 27 passes the traffic area was considered failed. The rut depth measured at failure was 3 1/8 inches. Photo C-52 shows the large cracks that were observed on both sides of the tire print.

On the fifth pass of the skid test, the load cart rutted the pavement to a depth of 5 inches. Photos C-53 and C-54 depict the traffic area before traffic and after failure.

C. TEST RESULTS

Field and laboratory test results are discussed in the following paragraphs.

1. Thickness, CBR, Water Content, and Density Data

After the completion of traffic tests, trenches were excavated across the traffic lane. After removal of the pavement surface, tests were conducted on each successive layer including the subgrade. These tests consisted of CBR, density, and water content. However, on two test features, WP-2 and W-1, free water was encountered before the depth of the subgrade was reached, making further testing impossible. Therefore, the test trench was backfilled. The procedure described above was followed in all the test trenches at both Wright-Patterson AFB and Whiteman AFB.

A summary of test results obtained from the seven test trenches is shown in Table A-8. The laboratory CE-55 density values shown in the table were at maximum density and optimum water content. The laboratory compaction curves and CBR data are depicted in Figures B-20 through B-29.

2. Laboratory Tests and Analyses on the Bituminous Surface Material

Samples of the bituminous surface material were obtained from each test feature. A summary of the laboratory test data, including asphalt content, stability, flow, voids, and density is shown in Tables A-9 and A-10. These data were obtained from tests following Corps of Engineers test procedures. Aggregate gradation curves of the asphalt-concrete mixture from each test feature are shown in Figures B-30, B-31, and B-32.

3. Traffic Tests

During trafficking, several methods were used to monitor the condition of the pavement surface. One method was by visual observations. At various pass levels, the load cart would be stopped, and measurements of the rut depths would be taken and recorded. Another method used for recording pavement conditions was the Pavement Condition Index (PCI) survey (Reference 8). A PCI rating of the pavement generally was not assigned, other than before traffic because of the very small area of the test features (300 square feet). AFR 93-5, (Airfield Pavement Condition Survey Report, Chapter 3)(Reference 8), states that a pavement feature is first divided into sample units. A sample unit for AC is an area approximately 5,000 square feet. However, the PCI did provide another method to evaluate the pavement performance during the traffic tests. The survey also depicted the progression of the distress types to the pavement and their severity levels with the increase in the number of passes.

The predominant types of pavement distresses and the corresponding densities for each test feature are summarized in Table A-11.

D. ANALYSIS OF RESULTS

Seven shoulder and overrun pavements located in environmental conditions similar to where ALRS pavements are to be built were trafficked to failure with an F-4 load cart. Pertinent structure and traffic test results are presented in Figures B-33 and B-34.

Pavements varied in age from 9 to 30 years. Maintenance in the form of seal coats or overlays had been performed on five of the seven pavements from 7 to 18 years after construction. The surface condition of the pavements ranged from excellent to poor using the PCI procedure.

The performance of the DBST (W-1) would not be acceptable for an ALRS pavement since foreign object damage (FOD) potential was unacceptable at only 100 passes. Structurally, the aged pavement performed adequately (280 passes to a 3-inch rut depth) but the surface became granular.

Two procedures were used to compare the structural performance of these pavements under the F-4 load cart. Initially, the CBRs of the pavement layers measured were input to the CBR design procedure. Comparison of predicted values and traffic results are shown in Figure B-35. Five of the seven pavements failed (1-inch rut depth) after nearly or equal to those predicted. In two cases (WP-4 and W-2) the predicted passes to failure were

greater than the results from traffic tests. The W-2 pavement contained 50 percent alligator cracking before traffic; therefore, the performance would be expected to be less than predicted. Although variation is noted between predicted and actual passes, the amount of variation is reasonable for pavement performance.

The pavement performance was also analyzed from a design viewpoint. The gradations of the base course materials (Figures B-18 and B-19) were compared to the specifications for graded-crushed-aggregate base course (Reference 7). The gradations are slightly outside the required limits. For example, the percent finer by weight passing the number 200 sieve ranges from 11 to 13 percent for Whiteman AFB pavements (Figure B-19). The specification range is 0-10 percent. A deviation of one to three percent appears reasonable for pavements 22 to 30 years old. The dry densities measured on the seven pavements ranged from 96 to 102 percent of the maximum density from the CE-55 compaction density. Again, the small deviation from the specification requirement seems reasonable for pavements from 10 to 30 years old.

Assuming that the base course meets the specified requirements for a 100 or even 80 CBR base course, the comparison to evaluated loads based on in-place subgrade CBR is given in Figure B-36. In every case, the performance was less than the estimated design traffic levels. The variation between predicted and actual passes to failure is unacceptable for design of marginal pavements such as ALRS.

Noting that the failure criterion for the CBR design procedure is a 1-inch rut depth, whereas failure for ALRS pavements is defined as a 3-inch rut depth, the variation is still unacceptable.

The reason the strength of the base courses of a majority of these pavements is below the design strength appears to be a high in-place water content. Water content samples taken at the surface of the base course were generally less than or equal to the optimum from the CE-55 laboratory compaction curves. Those at lower depths (see Table A-8 for WP-1, WP-2) were higher than optimum. Laboratory compaction results indicate for base courses of WP-1, WP-2, WP-3, W-1, W-2, and W-3 (see Figures B-20, B-21, B-22, B-26, B-27, and B-29) the CBR will be less than 100 for water contents above optimum. Of these pavements, an in-place CBR of 100 was measured only on the base course of W-2.

Testing of these pavements was conducted in July and August when the water contents of the layers would be expected to be lower when compared to winter or spring. The water contents of the subgrades were also much higher than optimum, indicating a source of water.

E. CONCLUSIONS

Based on field testing of seven pavements, the following conclusions are presented.

1. Pavements WP-2, WP-4, and W-1 exceeded 150 passes of distributed traffic before producing 3-inch ruts. The properties of these pavements were as follows:

	Surface Layer		Base			
Item	Type	Thickness	Type	CBR	Thickness	Pavement Age
WP-2	AC	3 in.	GP-GC	33	16 in.	24 yr, OL at 12 yr
WP-4	AC	2 in.	GW-GM	72	14 in.	11 yr
W-1	DBST	1 in.	GW-GM	33	14 in.	22 yr

2. Pavements tested which produced 3-inch ruts in less than 150 passes generally had a low CBR (less than 13) in the base course or at least in the lower half of the base course due to high water contents.

3. Old 2- and 3-inch AC pavements were found to be capable of sustaining the ALRS mission if the base course under the wearing surface was adequate.

4. A DBST will not be an acceptable surfacing for ALRS because of FOD potential.

5. For AC surface pavements designed for minimum traffic levels, the base course is the critical component. Strict requirements for water susceptibility, gradations, and in-place densities must be established and followed.

SECTION V

LABORATORY-ACCELERATED AGING OF ASPHALT MIXES

A. INTRODUCTION

In the field of asphalt paving, most pavements harden after the mixes are placed and exposed to the environment. The asphalt cement ages during heating, prior to mixing with aggregate, during mixing, and continues to age after placement of the asphalt mix. The primary causes of asphalt hardening (aging) are generally attributed to loss of volatiles, oxidation by air and water, and exposure to light.

This phenomenon of asphalt aging has been of interest to pavement engineers for many years. Many physical tests have been developed to characterize asphalts and index their behavior; viscosity, ductility, and thin-film oven tests are a few. Among these is the penetration test which has been used as both a classification test when asphalt cements were classified by penetration, and as an indicator of asphalt aging when its unaged penetration was known. It is within the context of aging that asphalt penetration was used in this study.

1. Purpose and Scope

The purpose of this study was to develop an accelerated laboratory aging test for asphalt mixes and evaluate the performance of various mixes subjected to the test. The scope of investigation was limited to asphalt aging or hardening as indicated by penetration test results on asphalt recovered from aged laboratory mixes.

2. Previous Asphalt-Aging Tests

References 9 through 14 describe laboratory aging tests performed on asphalt mixes during the past 30 years. As indicated in Table A-12, most of the methods are destructive because the aged specimens are destroyed to recover the asphalt for penetration testing. An indirect nondestructive method of determining aging was discussed in Reference 13.

Field correlations were made with laboratory tests of References 11 and 14. They indicated that a laboratory-accelerated aging test performed at 150°F for one week was roughly equivalent to about 1 year of field exposure in New Mexico.

3. Asphalt Penetration Changes and Proposed Hardening Indices

General practice in studies of asphalt mixes is to extract and recover the asphalt prior to performing penetration and other tests on the binder. When this is done, further tests can be conducted on the asphalt to determine its physical properties such as penetration.

Figure B-37 illustrates generalized penetration test results as a function of mix age. Also shown are asphalt hardening components due to

heating, mixing, and aging. If asphalt penetrations can be determined at the points indicated, assuming the initial penetration is known, indices can be computed for the components of asphalt hardening. This is shown in Figure B-38 along with indices proposed to quantify hardening components of the asphalt mix.

The heating and mixing index quantifies that portion of hardening which occurs during mix manufacturing processes. This penetration loss after field mixing has been shown to range from 5 to 46 percent (References 11 and 14).

The retained penetration index is simply the ratio of the recovered asphalt penetration to the original asphalt penetration. Immediately after a mix is produced, this index has its highest value; as the mix ages, the index decreases.

Age hardening is indicated by the aging index. As shown in Figure B-38, this index equals one minus the sum of the other two indices. As noted, the aging index increases with exposure to the environment. Due to this, it is reasonable to expect the index to directly reflect the degree of asphalt age hardening.

B. MATERIALS, PREPARATION AND EQUIPMENT

Materials, specimen preparation, and equipment were generally typical of hot mix design procedures. Some differences were the use of a gyratory compaction machine and ovens to age compacted mixes.

1. Aggregate and Asphalt Cement

Aggregate types used in preparing asphalt mixes were a crushed limestone from Alabama and a locally available crushed chert gravel. Each aggregate was nonabsorptive with less than 2 1/2 percent water absorption. A natural sand was also used in some of the mixes. Aggregate had been previously separated by sieve size and were blended to produce gradations shown in Figure B-39 and Table A-13. Gradations A, B, and E are quite similar and approximate the median of the 3/4-inch maximum surface course gradation required for high contact pressures (Reference 15). Gradation C approximates the lower limit of the 3/4-inch maximum size gradation for surface courses with high contact pressures. Gradation D represents the upper limit of the 3/4-inch maximum surface course gradation for high contact pressures. Gradations A, B, and E were referred to as "medium," Gradation C was "coarse," and Gradation D was "fine."

The Fineness Modulus (FM) which was used to quantify gradation (Reference 16) is shown in Table A-13. FM is a term that is used to characterize concrete aggregates.

Properties of the three asphalt binders used in this study are given in Table A-14. These asphalt binders consisted of two AC 20 grade asphalts and an AC 5 grade. The AC 5 and one of the AC 20 grades were produced by

Southland Oil Company of Yazoo City, Mississippi. The other AC 20 was produced by Texaco in Virginia.

Mixes are identified by aggregate type, gradation, and asphalt producer and type. Typical identification includes the following:

<u>Aggregate</u>

LS - Alabama Limestone
Gravel - Chert Gravel

<u>Gradation</u>

Coarse - Gradation C
Medium - Gradations A, B, and E
Fine - Gradation D

<u>Asphalt</u>

S - Southland
T - Texaco

2. Specimen Preparation

Aggregate was blended into approximately 1200 gram batches, dried to constant weight at 300°F and mixed with asphalt binder heated to about 270°F. This produced sufficient mix to form 4-inch diameter by 2 1/2-inch high Marshall specimens.

A compactive effort comparable to the 75-blow hand hammer was applied to the specimens with a gyratory compactor (Reference 17). A vertical pressure of 200 psi was applied while specimens were gyrated through a 1-degree angle for 30 revolutions. This compaction effort was applied at a mix temperature of 250°F.

Stabilities, flows, and mix properties were determined in accordance with Methods 100 through 102 of Reference 18, and ASTM D 1556 of Reference 19.

3. Equipment

Equipment used in this study consisted of typical laboratory equipment used in the design and analysis of bituminous paving mixtures. Reference 18 lists some of the apparatus required. However, since the asphalt binder was recovered and tested after aging, additional equipment and procedures, as indicated in Reference 20, were required.

For the development of an aging test, three mechanical convection ovens were used. Each was capable of temperatures ranging from 95 to above 600°F. In the temperature range used (140 to 275°F), the interior oven temperature was uniform to ± 2°F.

C. GENERAL AGING TESTS

The first series of tests was run on limestone aggregate mixes with 5.4 percent Southland AC 20 and an "A" gradation (medium). Approximate properties of the mix were:

Unit Weight, pcf	151.6
Stability, pounds	2070
Flow, .01 inch	12
Air Voids, percent	3.5
Voids Filled, percent	78

Marshall specimens were prepared and aged for varying periods at one of three temperatures. Each oven maintained a constant temperature, either 140, 225, or 275°F. Specimens were aged in this accelerated mode for 1, 3, 7, 14, and 28 days. Asphalt penetration tests were performed on the asphalt obtained from unaged specimens and from aged specimens. A temperature-controlled and mechanically stirred kettle was used to heat the asphalts to mixing temperature. Table A-15 summarizes penetration test results.

Figure B-40 shows retained penetration index and age relationships for the three test temperatures. The plotted data in conjunction with Table A-15 data indicates a heating and mixing index from 27 to 56 percent which is quite high (and within limits given in References 12 and 14). Personnel conducting the tests indicated that the asphalt had been reheated several times before this testing began. However, data of Table A-15 was suitable for purposes of indicating the general effect of temperature and exposure on asphalt penetration.

1. Specific Aging Tests

The laboratory aging test was developed to artificially age asphalt concrete mixes over a short period of time to produce asphalt binder properties similar to that experienced in the field after aging for 10-15 years. Most asphalt technologists agree that an asphalt concrete usually requires some type of maintenance or rehabilitation after the asphalt binder has decreased to a penetration value of 20. Three oven temperatures, 140°F, 225°F, and 275°F, were evaluated for aging the asphalt concrete samples. Sample mixes were prepared, aged for various times, and testing to determine the asphalt penetration. The test results presented in Table A-15 indicated that aging at 225°F would reduce the penetration of an AC 20 asphalt cement to approximately 20 after 7 days. Aging at 140°F would not produce the desired results after 30 days and therefore was considered unacceptable due to the long aging time required. Aging the asphalt concrete mixes at 275°F produced the desired asphalt cement properties after one day; however, at this high temperature, distortion of the samples was observed. Heating the mixes in an oven at 225°F for seven days was the most satisfactory procedure for accelerated aging.

A review of test results obtained from samples of asphalt concrete at Whiteman AFB and Wright-Patterson AFB indicates that the selection of asphalt penetration of 20 for laboratory aging is reasonable and representative

of an aged pavement requiring maintenance. The asphalt cement recovered from samples taken from Wright-Patterson was approximately 20 penetration for all four pavement features, whereas the penetration of the asphalt cement recovered from the asphalt concrete samples taken from Whiteman AFB was significantly below 20 for both features. The low asphalt penetration at Whiteman could be explained by several factors which include: low initial asphalt penetration, use of asphalt cement with poor aging characteristics, and/or exposure to a climate that facilitates rapid aging. The test properties of samples from Whiteman and Wright-Patterson clearly indicate that the asphalt concrete in each of the features was representative of aged asphalt concrete mixtures.

For the remaining tests of this study, care was taken to insure that the asphalt cement was not reheated more than one time prior to mixing with the aggregate.

For this part of the study, six different mixes were produced. Each mix consisted of a specified aggregate type, aggregate gradation, asphalt type and asphalt source. Each mix was produced at three asphalt contents by weight (see Table A-16). Five specimens were made at each asphalt content; two were subjected to accelerated aging and three were not aged.

Table A-16 gives density, voids in the total mix, voids filled with asphalt, stability, and flow summary of aged and unaged mixes. As shown, flow values of the aged specimens were generally higher than those of the unaged specimens. This was probably caused by plastic movement, slight changes in the shape of the specimens during exposure to the aging environment. Initial loading of the aged specimens in the Marshall machine probably reshaped any out-of-roundness, thus giving higher flow values. Higher stabilities are expected in aged pavements.

Asphalt cement was recovered from the mixes (a destructive process) and penetration tests conducted on the asphalt. This penetration data was used to compute asphalt indices for each of the six mixtures and the results are shown in Table A-17.

2. Heating and Mixing Index

The heating and mixing index was measured for each of the mixtures. It was believed that the heating and mixing index was dependent on aggregate type, aggregate gradation, asphalt type, asphalt source, and many other properties.

Figure B-41 shows heating and mixing index data for mixes that were not aged. Since more data were obtained on limestone and Southland AC 20 mixes, these were used to show the general trend in the data. The figure indicates that a medium gradation with a fineness modulus in the range 3.95 to 3.97 minimized heating and mixing asphalt hardening.

Ranking of mixes in terms of increasing asphalt hardening during heating and mixing is shown in Table A-17 and listed below:

a. Limestone and Texaco AC 20
b. Gravel and Southland AC 20
c. Limestone and Southland AC 5
d. Limestone and Southland AC 20

Gravel mixes required higher asphalt contents to produce the same aging index as limestone mixes with the same type asphalt. A possible explanation is that the gravel aggregate is tougher to compact, thus producing more voids in the mineral aggregate in the compacted mixture than those produced in limestone mixtures.

3. Aging Index

Individual aging indices were analyzed with respect to variables such as gradation, aggregate type, asphalt type, and mix properties such as air voids, asphalt content, etc. Linear regression analysis was used to aid in the analysis. Asphalt content by volume was found to provide the highest and best correlation with aging index.

For the series of mixes made with limestone and the Southland AC 20, Figure B-42 illustrates that asphalt content by volume has a great effect on the aging index of the asphalt mixtures.

Figure B-43 shows aging indices for all mixes studied. The effects of aggregate and asphalt types are easily seen. The limestone and both AC 20 asphalt cements produced mixes that minimized the aging component of hardening at lower asphalt contents.

4. Retained Penetration Index

Retained penetration indices, the ratios of recovered asphalt penetration to original penetration, are shown in Figure B-44. Mixes produced with limestone aggregate and Texaco AC asphalt cement were generally most resistant to the effects of asphalt hardening. Limestone mixes produced with Southland asphalts of differing grades showed almost identical behavior; this is probably due to similarity of petroleum base and refining processes and techniques.

Asphalt content by volume again showed the best correlation with overall asphalt hardening as indicated by the retained penetration index.

D. PRELIMINARY CRITERIA FOR MINIMIZING ASPHALT HARDENING

Analysis of data on the aged and unaged mixes indicated that criteria for the selection of an optimum asphalt content for an age-resistant mix would be different than those criteria currently in use (Reference 15). Aged and unaged stability and flow data, aging indices, and retained penetration indices were examined to set preliminary criteria based on the 7-day accelerated aging test at 225°F.

Based on test data, the following laboratory criteria are presented:

Void requirement of 2-4 percent
Unaged flow less than or equal 16
Minimize aging index
Retained penetration index of at least 0.50

Current practice for bituminous mix design is based on regular frequency of pavement use while an alternate launch and recovery surface is assumed to be used irregularly and to stand idle most of the time. The voids requirements were reduced to improve durability. The second criterion is the same as current practice. This requirement will assure that the surface will not have a great tendency to displace under loads.

Minimizing the aging index, based on the accelerated aging test, should minimize the effect of asphalt hardening and decrease long-term brittleness. At this point in the use of aging indices, there were not enough data to quantify or place a number on a minimum aging index.

The retained penetration index of 0.5 can be used to check adequacy of the mix at a design asphalt content. This value was determined by application of the first three criteria to the data of this study and noting the minimum retained penetration index.

A review of the data in Tables A-16 and A-17 indicated that of the mixes tested, several met the criteria. The acceptable mixes are listed below:

Medium-graded limestone and Southland AC 20 asphalt at 5.5 percent by weight (13.0 percent by volume).

Fine-graded limestone and Southland AC 20 asphalt at 6.5 percent by weight (15.2 percent by volume).

Medium-graded limestone and Texaco AC 20 asphalt at 5.0 and 5.5 percent by weight (11.8 and 13.0 percent by volume).

Medium-graded gravel and Southland AC 20 asphalt at 8.0 percent by weight (17.8 percent by volume).

Coarse-graded limestone and Southland AC 20, 5.5 percent by weight (13.0 percent by volume).

Medium-graded limestone and Texaco AC 5, 5.5 percent by weight (13.0 percent by volume).

Instances will occur when either there will not be a choice between asphalt types or producers, or aggregate type, or when facilities are not available to perform accelerated aging tests. In either case, current guide specifications, such as "Guide Specifications for Military Construction - Bituminous Intermediate and Surface Courses for Airfields, Heliports, and Tank Roads (Central - Plant Hot-Mix)," will have to be modified.

New sections and/or notes, discussing mix composition and construction to minimize asphalt age-hardening, should be made an optional part of the guide specifications. They should present previously discussed preliminary mix criteria or any future revisions to the criteria, and provide general guidance on field construction of this type of asphaltic concrete. Field construction comments should address the need for determining basic physical property data, such as voids and density, that could be readily used as an indirect indicator of relative mix-hardening potential.

In those instances where aging tests have not been performed on an asphalt mix prior to construction, a maximum air void limit on the field-compacted mix of 6 percent should retard the rate of asphalt hardening.

E. SUMMARY

1. An accelerated aging test was developed and performed on several mixes. Its 225°F temperature exposure for 7 days should represent the equivalent of several years of natural exposure. Data indicated the procedure to be equivalent to 8-15 years exposure at Wright-Patterson AFB.

2. Asphalt-hardening indices were developed and used as indicators of asphalt aging or hardening. Penetration test results were used to indicate asphalt hardening. Heating and mixing index indicates asphalt hardening due to combined effects of heating asphalt cement prior to mixing (with aggregate) and the mixing operation. The aging index represents environmental hardening due to exposure. Finally, the retained penetration index represents the general relationship of asphalt hardening ratio between an aged asphalt in a mix and the virgin asphalt. Asphalt was recovered from mixes after the mixing process and after accelerated aging and penetration test results were evaluated with respect to initial penetration values.

3. Based on mix properties of both aged and unaged specimens, preliminary criteria for mix design of age-resistant pavements were presented.

F. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are made:

1. Aging index or degree of asphalt hardening caused by aging was found to be inversely related to asphalt content by volume. In other words, the amount of aging decreases as the asphalt content increases.

2. The use of higher penetration asphalt cements results in mixes with high penetrations after aging.

3. The source of asphalt cement does affect the performance of asphalt mixtures when subjected to the oven-aging test.

4. A change in aggregate gradation does not directly affect the oven-aging performance; however, a change in gradation usually requires a change in asphalt content which does affect aging performance. A gradation should be selected (within limits) which requires the maximum asphalt content to be added to the mixture.

5. The air voids in a mixture does affect the aging performance. The air voids should be minimized (within limits) by use of more asphalt cement or improved compaction of mixture.

The following recommendations are made:

1. The present requirements for airfield mixtures which require 3-5 percent voids in the total mixture should be changed to require 2-4 percent. The voids filled with asphalt requirements which now require 70-80 percent should be changed to 75-85 percent. These requirements are for mixes made with nonabsorptive aggregates.

2. The present gradation requirements should be adopted for the ALRS asphalt mixtures. The requirement for the amount of material passing the number 200 sieve is 3-6 percent. It is desirable to maintain the amount of material passing the number 200 sieve on the low side of these requirements.

3. If two or more sources of asphalt cement are available, these asphalt cements should be tested in the laboratory to determine their aging characteristics before selecting the final source.

4. The amount of voids in the in-place (field-compacted) AC should not exceed 6 percent.

5. Additional accelerated aging tests, based on the method and indices developed in this study, should be conducted on a broader range of aggregate types and asphalts, including absorptive aggregates.

6. Field correlations should be made based on laboratory aging test results. Test sections should be designed and constructed based on laboratory work. Later, periodic testing of field mixes can be done to correlate field aging with laboratory aging.

SECTION VI

FROST DESIGN PROCEDURE FOR THE ALRS PAVEMENTS

A. INTRODUCTION

In many areas where ALRS are necessary, the pavement system and subgrade soils will experience seasonal freezing. The upper layers of the pavement system may also experience several freeze-thaw cycles during the winter. Therefore, the pavement system must be designed to preclude excessive roughness due to frost heave. It must also be capable of sustaining the design traffic loads and number of coverages during periods of reduced subgrade strength due to thaw weakening.

This section contains information and data on three primary subjects:

1. Design and mean air freezing indexes for selected locations;

2. Results from laboratory frost-heave tests on soils from Air Force bases where the traffic tests discussed in Section IV were conducted; and

3. Thickness requirements for several typical pavement cross sections which may be used for ALRS.

B. FREEZING INDEX

The air freezing index is a measure of the severity and duration of subfreezing temperatures observed during a freezing season. A numerically larger freezing index generally results in a greater depth of frost penetration and potentially greater frost heave.

If a significant thickness of the subgrade soil becomes frozen, the depth of frost penetration may not be extremely important when the soil loses its strength upon thawing. On the other hand the prior frost depth may be important during thawing because a frozen layer beneath the thawed soil will restrict, or eliminate, downward vertical drainage. As a result the thawed soil may remain in a weakened condition until the previously frozen soil is entirely thawed.

The first phase of this study was to develop design air-freezing index values for 13 locations in the United States, Germany, and Korea. The design air-freezing index is the average of the three coldest winters in the last 30 years using the computational method in Reference 21. If 30 years of record are unavailable, the maximum air-freezing index for the latest 10-year period may be used or the design freezing index may be based on an intermediate period of record longer than 10 years but less than 30 years. The latter alternative was used in this study, as described below.

The 13 locations where air-freezing indices were determined are shown in Table A-18. The sites in Germany and Korea are candidates for ALRS and the US bases provide environmental conditions, i.e. design freezing indices approximately equal to those observed at the foreign bases. Field studies

discussed in Section IV were conducted at two of the US bases where air-freezing indices were computed. Data in the table also indicate the period of the temperature record at each location, the mean air-freezing index and the largest, second largest, and third largest air-freezing index at each location. The last column in the table illustrates the design freezing index recommended for each site. Wright-Patterson Air Force Base, Ft. Devens, Bitburg Air Base and Kusan Air Base are the only locations where at least 30 years of air temperature records are available. At these four locations the average of the three coldest winters was used to compute the design freezing index. At all of the other sites, the average of the two coldest winters was used to obtain the design air freezing index. Design air-freezing indices in Table A-18 are larger than those listed in Reference 1 for many of the same sites. Since data in Table A-18 were obtained from air temperature measurements made at the base or from a nearby site where air temperatures were measured, we are confident that these data represent conditions at the site. We do not know how the German Ministry of Defense developed the design indices presented by Barker, et al (Reference 1), and therefore cannot explain the exact causes of the differences. Some possible reasons for the differences are: a different method of determining design air freezing indices, use of data from locations not representative of the air bases, and use of data from a different time period.

The six sites in Germany and the two sites in Korea are also discussed in Reference 1. The sites in the United States were chosen by the U. S. Army Engineer Waterways Experiment Station and Air Force personnel to provide test areas having environmental conditions similar to those in Germany and Korea.

The smallest design air-freezing index of the locations studied is at Zweibrucken Air Base, Germany. The largest design air-freezing index is at the Seneca Army Depot in New York. The freezing index at Whiteman Air Force Base, Missouri, is approximately equal to indices at Hahn Air Base, Ramstein Air Base and Sembach Air Base in Germany.

Wright-Patterson Air Force Base, Ohio, has a design air-freezing index similar to that of Osan Air Base, Korea. None of the bases in the United States which were included in this study experiences the mild design winters exhibited at Bitburg Air Base and Zweibrucken Air Base in Germany or Kunsan Air Base in Korea.

C. FROST-SUSCEPTIBILITY TESTS

The standard frost-susceptibility test used by the U. S. Army Corps of Engineers consists of a single cycle of frost penetration into the 6-inch diameter by 6-inch long sample (Reference 22). Duplicate samples of each material are frozen and then the samples are normally cut or fractured longitudinally after freezing. One part is retained to observe, measure, and photograph ice distribution and the other is normally cut or broken into horizontal layers approximately 1-inch thick to quantify the water distribution after freezing. For this study, the procedure described above was followed for one sample and the second sample was allowed to thaw and drain at a temperature of about 70^{o}F for 12 to 24 hours; at that time a laboratory

California Bearing Ratio (CBR) test was conducted on the sample. After the CBR test, the second sample was divided into 1-inch-thick layers and the moisture content of each layer was obtained.

The grain-size distribution curves for the soil material, taken from the test features at Wright-Patterson AFB are shown in Figure B-18. The materials labeled WP-1 (a base course material) and WP-3 (subgrade) were used to conduct the standard frost-susceptibility tests.

Figure B-19 contains the grain-size-distribution curves for the soil material obtained from the test locations at Whiteman AFB. Standard frost-susceptibility tests were conducted on the subgrade soils labeled W-2 and W-3. Duplicate samples of each material were also frozen and then sectioned to obtain moisture content distribution.

Table A-19 contains results from all of the laboratory frost-heave tests, and Table A-20 shows CBR results after thawing. Since all of the soils tested contain a significant amount of clay, results from the laboratory frost-heave and thawed CBR tests would probably have been different if the samples were subjected to two or three freeze-thaw cycles. The additional freeze-thaw cycles would have probably caused higher heave rates and lower CBR values after thawing.

None of the materials from Wright-Patterson AFB heaved significantly. The frost-susceptibility classification ranged from low to negligible.

The soils from the subgrade samples from Whiteman AFB also heaved very little. Frost-susceptibility classification of the four samples ranged from negligible to very low.

D. THICKNESS REQUIREMENTS

Frost heave may occur during the winter resulting in deformed pavements. During freeze-thaw cycles and especially during spring thaws, the supporting capacity of pavements may be severely weakened. Therefore, in seasonal frost areas the thickness design process for pavements must consider two effects of frost action. Two methods are used to determine the thickness of pavements which will provide adequate resistance to distortion by frost heave, and cracking and distortion under traffic loads caused by seasonal reduction of supporting capacity. The two methods are the limited subgrade penetration method used primarily to control pavement roughness; and the reduced subgrade strength method which provides adequate strength in the pavement, base, and subbase to carry the design traffic when the subgrade is severely weakened during spring thaws. When the design air-freezing index exceeds about 300 to 400°F-days, the reduced subgrade strength procedure will generally provide the least expensive design for loads corresponding to the F-4 and F-15 aircraft.

Based on recommendations outlined in this report the following two requirements for ALRS pavements have been established: (1) minimum thickness of asphalt concrete surface is 2 inches, and (2) the upper 6-inch layer of base course will have a minimum 100 CBR.

1. Base Course and Subgrade Preparation

Base course and subgrade preparation requirements in Reference 21 must be met also. These requirements are summarized below:

a. At least 4 inches of free-draining material shall be placed directly beneath the lower layer of bound base or the paved surface. Material in the 4-inch layer must meet base course requirements and contain no more than 2 percent by weight passing the number 200 sieve.

b. The top 50 percent of the total thickness of granular unbound base must be non-frost-susceptible (NFS) material containing not more than 5 percent by weight passing the number 200 sieve.

c. The lower 50 percent of the total thickness of unbound base may be either NFS material or S1 or S2 material.

d. If subgrade freezing occurs, the lower 4-inch layer of base course or subbase which is in contact with the subgrade must meet the requirements for a filter over the subgrade. Alternatively, a geotechnical fabric may be used as a filter.

e. The depth of subgrade preparation is governed by the design freezing index and the thickness of pavement, base course, and subbase. The depth of preparation, i.e. mixing and blending, is the lesser of 24 inches or, for ALRS pavements, two-thirds of the pavement, base course, and subbase thickness required to prevent the subgrade from freezing.

f. Stabilized layers may be used but they must meet durability tests required in Reference 21. In addition, when Portland cement, lime or lime-cement-fly ash stabilizers are used, they must be placed sufficiently early in the construction season to allow adequate strength development before subfreezing temperatures occur.

g. Thermal insulating layers may be used to protect the subgrade from freezing or to limit the penetration of frost into the subgrade.

h. Membrane-encapsulated soil layers (MESL) may be used to limit or prevent frost action in the subgrade.

2. Limited Subgrade Frost Penetration Method

This design procedure is used to control pavement roughness due to frost heave by limiting the depth of frost penetration into the frost-susceptible subgrade. When unusually high standards of surface smoothness are required, or when thermal insulation is used, prevention of frost penetration into the subgrade may be desirable, thus allowing complete

protection of the subgrade from freezing. If thermal insulation is used, complete protection may be economical; otherwise, the extra cost of providing complete protection will not be appropriate for ALRS pavements.

Figures B-45, B-46, and B-47 may be used to determine the pavement thickness when applying the Limited Subgrade Frost Penetration method. Figure B-45 or B-46 is used to compute the depth of pavement, base course, and subbase required to completely protect the subgrade from freezing. This depth is then used with Figure B-47 to determine the thickness of pavement, base course, and subbase which will acceptably limit frost penetration into the subgrade.

Data from Figure B-45 indicate that if the design air freezing index is 300^{o}F-days, the average dry density of the base and subbase is 135 pounds per cubic foot and the average water content of the base and subbase is 7 percent, the thickness of pavement, base, and subbase required to prevent freezing into the subgrade is 26 inches. Since the pavement thickness is 2 inches, the thickness of base and subbase to prevent freezing into the subgrade is 24 inches. This thickness is then used in Figure B-47 with the ratio of subgrade moisture content to the moisture content of the base and subbase (r-value) to obtain a design base thickness. If the average water content of the subgrade is 14 percent, the r-value is 14/7 = 2. Thus the necessary combined thickness of base and subbase is 16 inches. Frost penetration into the subgrade would be 4 inches in the design winter. If other reasonable combinations of base and subbase densities and moisture contents and subgrade moisture contents are used, the design combined base and subbase thickness will be about 14-16 inches for a design air-freezing index of 300^{o}F-days. Greater design air-freezing indices will result in greater required combined thicknesses of base course and subbase.

Figures B-48 through B-53 were prepared primarily to estimate the depths of frost penetration which could be anticipated beneath various types and thicknesses of pavements designed for the ALRS. Figures B-48 and B-49 relate the surface freezing index and frost penetration for other thicknesses and densities of base course. For asphalt concrete pavements without insulating layers, the modified Berggren equation (Reference 23) was used to compute the frost depths in the figures. For asphalt concrete pavements without insulating layers, air-freezing indices were multiplied by 0.75 to obtain surface freezing indices. A 3-inch thick pavement was assumed for the calculations. Although the current ALRS design requires a 2-inch thick pavement, the difference in maximum seasonal frost penetration beneath 2-inch and 3-inch thick pavements is insignificant. Data in Figures B-48 and B-49 are not to be used for design purposes, but are presented to illustrate the maximum depths of frost penetration beneath pavements having different base course properties and thicknesses. For a given freezing index the total frost penetration depths are greater for thicker base courses. The frost depths are also greater for the high density, low-moisture content material in Figure B-48 than the lower density, higher-moisture content gravel in Figure B-49.

Figure B-50 illustrates frost penetration depths beneath asphalt concrete pavements placed directly on membrane-encapsulated soil layers (MESL) of different thicknesses. Since the thermal energy required to freeze the water in 1 cubic foot of MESL is only slightly less than the thermal energy required to freeze the water in 1 cubic foot of the subgrade soil, the thickness of MESL having thermal properties used in this example has little effect on the total depth of frost penetration.

Figures B-51, B-52, and B-53 illustrate the effects of different thicknesses of extruded polystyrene insulation and gravel on the depth of frost penetration. Note that for insulated pavements the surface-freezing index is 90 percent of the air-freezing index, compared to 75 percent for uninsulated pavements. Frost penetration depths in Figure B-49 and Figures B-51 through B-53 can be compared to evaluate the effect of using an insulating layer beneath a layer of gravel. The ameliorating effect of insulating layers on frost penetration depths is evident for freezing indices greater than 100-300°F-days, depending on the thickness of gravel above the insulation.

3. Reduced Subgrade Strength Method

This design procedure was developed to assure that pavements in seasonal frost areas would contain a sufficient thickness of pavement and base course to sustain the design traffic during thaw weakened periods.

This design procedure for flexible pavements is the same as the CBR design procedure used in pavements not affected by frost action with one important difference. In seasonal frost areas each soil type is assigned to one of four frost-susceptibility classification groups. Each of these groups is assigned a frost-area soil support index (FASSI) (Table A-21) which is used like a CBR in designing flexible pavements.

It is extremely important to note, however, that the FASSIs are effective weighed values averaged over the entire life of a pavement. <u>The FASSIs are generally not the minimum CBR values experienced during frost-melting periods</u>. Since all the traffic on an ALRS could occur over a time span of only a few hours or a few days, the frost area soil support indices may not provide pavement thicknesses sufficient to sustain the design traffic if it should occur during periods of subgrade thawing in the winter and spring. For example, a lean subgrade classified as an F4 soil under the frost classification system may exhibit a normal period CBR of 6. The frost area soil support index for this material is 3.5, but if its CBR were only 2.0 after thawing, pavements designed with the FASSI may fail due to inadequate thickness. The pavement thickness for a frost-area soil support index of 3.5 is 14.5 inches (Figure B-54) when the aircraft gross weight is 60,000 pounds and the design traffic is 150 passes. For the same aircraft and amount of traffic, a pavement thickness of 18.5 inches is required over a subgrade having a CBR of 2.0.

A question which may be raised is that if ALRS pavements are designed using the frost-area soil support index for a particular soil and the CBR of the soil during application of the design traffic is less than the FASSI, what is the risk of pavement failure prior to sustaining the desired amount

of traffic? The answer is that the probability of premature failure of the pavement is high unless the gross weight of the aircraft is reduced. In the example given on the previous page, the gross load of the aircraft would have to be reduced to about 38,000 pounds if the CBR of the subgrade were 2.0 rather than the 3.5 for which the pavement was designed. The number of passes of the F-4 aircraft, having a gross weight of 60,000 pounds, before failure of the 14.5-inch-thick pavement is about 50.

Chamberlain (Reference 24) stated that the thaw period for in-service highway pavements studied by Schrivner (Reference 25) ranged from a few days to 2 weeks but the time for the pavement to reach a deflection which was only 20 percent greater than the fall deflection was 35 to 60 days. Unfortunately, no data are available to estimate the period of time when the CBR of the subgrade may be less than the FASSI. Also, no extensive correlation between laboratory and field CBR values after freezing and thawing has been conducted. The period of recovery from a thaw-weakened condition is influenced by the hydraulic properties of the soil. For example, a sandy silt will probably drain excess water and recover its strength more rapidly than a highly plastic clay because the clay has a lower permeability. When roads or airfield pavements are underdesigned, the road and airport managers must restrict traffic loads or, in extreme cases, close facilities to traffic for a period of a few days to several weeks in the spring. Typically these periods are 2 to 6 weeks long. It seems highly unlikely that a design premise of closure or restriction of traffic on the ALRS pavements during such periods could be acceptable to the Air Force.

Recently Chamberlain (Reference 26) has developed a laboratory frost-susceptibility test which includes a CBR test made subsequent to two freeze-thaw cycles and a CBR test. Behr (Reference 27) has advocated a similar test and the roads and highways in West Germany are now designed, using a laboratory test which includes freeze-thaw cycles and a CBR test. The laboratory frost-susceptibility test developed by Chamberlain is conducted in a severe environment for the soil. The soil specimen is 6 inches long and a water supply is maintained at the base of the sample. The CBR obtained in this test may be lower than the minimum CBR exhibited by the same soil in the field unless the water table is near the top of the subgrade. On the other hand, repeated freeze-thaw cycles on actual pavement systems may, under particularly adverse conditions, result in CBR values similar to those measured in the laboratory test. The probability that the ALRS pavements will receive essentially no traffic to recompact the subgrade from year to year represents one such adverse condition.

Although the preceding argument would support a contrary position, the U. S. Army Cold Regions Research and Engineering Laboratory (CRREL) recommends that the tentative design of the ALRS pavements in seasonal frost areas be based on the recommended FASSI values listed in Table A-21. Reasons for their recommendations are: the FASSI values were developed using results from field test sections incorporating lower quality base course materials than currently specified; the low probability that the time of use will be coincident with a winter equal to or greater than the design freezing index;

the low probability that the time of design traffic will be coincident with the period when the subgrade is in its weakest condition; and the overall conservatism of the CBR design method.

Large-scale studies wherein traffic will be applied while the subgrade is in its weakest condition, during the thawing period, are planned. Results from these tests will indicate whether this tentative design is adequate. The planned tests include only one cohesive subgrade soil and three pavement cross sections. However, additional tests to determine the extent and duration of thaw-weakened conditions beneath in-service runways and taxiways are recommended. These tests should include a comprehensive combined field and laboratory study. Hydraulic and strength properties of the soils should be evaluated in the laboratory and on in-service pavements. Observations should include locations of frozen and thawed zones with time and pore water pressures (suctions) with time and depth. Field-strength tests on the pavements should be made with a falling weight deflectometer and testing should be conducted for at least 2 to 6 weeks while the soil is thawing and recovering in the spring. Perhaps a study of the performance of shoulder pavements during thawing periods would be of most benefit to the design of ALRS pavements. This additional information will reinforce any decision which may be made to reduce the FASSI values now recommended for ALRS pavements subjected to seasonal freezing and thawing.

Figure B-54 should be used for tentative design of ALRS pavements in seasonal frost areas. The required thicknesses of flexible pavements are listed in Table A-22. The thicknesses are based on FASSI values listed in Table A-21, and a gross aircraft weight of 6,900 pounds for 150 passes as shown in Figure B-54.

Subgrade preparation may also be necessary in some situations. Processing of the subgrade will include scarifying, excavating, blending, and compacting. As required by TM 5-818-2 (Reference 21), the required depth of subgrade preparation will be the lesser of either 24 inches or two-thirds of the frost penetration depth given by curves in Figures B-45 or B-46 minus the actual combined thickness of pavement, base course, and subbase. The depth of subgrade preparation is measured downward from the top of the subgrade.

For example, assume that the design air-freezing index at a hypothetical Air Force base is 600^{o}F-days, the average density and moisture content of the base and subbase are 135 pounds per cubic food and 7 percent, respectively, and the subgrade is a lean clay, F4. Frost would penetrate to a depth of 40 inches in a gravel embankment in this environment (Figure B-45). Two-thirds of this amount is 26.4 inches and the required thickness of flexible pavement, base, and subbase from Figure B-54 or Table A-22 is 14.5 inches. Therefore, the subgrade would be scarified and blended to achieve a more uniform condition for a depth of (26.4 inches - 14.5 inches) 11.9 inches.

E. CONCLUSIONS

Of the 13 Army and Air Force bases examined in this section, only Zweibrucken Air Base, Germany, is exposed to a design air-freezing index less than 400°F-days; its design freezing index is 231°F-days.

The CBR of the subgrade soil (CL) from Wright Patterson AFB after one laboratory freeze-thaw cycle was 4.8. The *in situ* CBR measured during the summer traffic testing ranged from 10.5 to 13.8. The laboratory CBR of this same material at optimum density and moisture content was 48.

The limited subgrade frost penetration design procedure may provide the most economical ALRS pavements at Zweibrucken Air Base, whereas at the other bases the reduced subgrade strength design procedure will provide the most economical pavement designs.

The minimum thicknesses of ALRS pavements and bases over subgrades including Frost Groups F1 and S1 is 8.5 inches; over F2 and S2 subgrades the minimum thickness is 10.5 inches; and over F3 and F4 subgrade soils the minimum thickness is 14.5 inches. If the normal period CBR is less than the FASSI for a particular subgrade soil, the normal period CBR will be used for design and will result in pavement thicknesses greater than these.

SECTION VII

ANALYSIS AND DISCUSSION OF RESULTS

A. GENERAL

In the design of airfield pavements, the two environmental factors of concern are temperature and moisture. Temperature and moisture are the primary variables that affect the material properties of pavement systems. Some of the distresses in AC surfaced pavements (Reference 28) related to temperature and moisture either singularly or in combination are:

- Potholes
- Loss of cover aggregates
- Raveling
- Weathering
- Alligator cracking
- Reflective cracking
- Shrinkage cracking
- Shoving
- Frost heave

With the exception of potholes, shoving, and frost heave, these distresses were observed in the pavements tested at Wright-Patterson AFB and Whiteman AFB. An actual ALRS pavement in Germany was surveyed for distresses. Results are presented below.

B. HAHN ALRS CONDITION SURVEY

During April 1982, a condition survey was conducted on an existing ALRS pavement at Hahn AB, Germany, for determination of type, amount, and severity of distresses. The pavement facility at Hahn is 5,200 feet long by 50 feet wide and was constructed during late Fall 1978. Construction weather conditions were very poor. The structure of the pavement (Figure B-55) consisted of 3 inches of AC over 15 inches of aggregate base. Material specifications are shown in Table A-23. The AC was placed in two layers. Samples taken during the survey indicated two layers of 1 1/2 inches in thickness which agrees with the material specifications but not the construction drawings (see Figure B-55).

The Pavement Condition Index (PCI) procedure (Reference 8) was used to evaluate the pavement. The average PCI for the feature was 86 or excellent. The predominant distresses located were block cracking (1.3 percent of area), and raveling or weathering (1.1 percent of area). Severity levels for distresses were low or medium. Other distresses in small amounts consisted of depressions and longitudinal and transverse cracking. Although a french drain was indicated on the construction drawing, there was standing water along the sides of this runway (Photo C-55). A structural evaluation was not made but the base was probably saturated and this would contribute to an early failure.

Samples of the AC surface along the edge of the pavement were collected and tested in the laboratory. Results are presented in Table A-24. The test results indicated that the recovered asphalt cement penetration was 64 in the surface and 104 in the binder course. The difference in the penetrations for the two layers is reasonable, since the surface course is exposed to the atmosphere and therefore ages faster. These penetration values are higher than normally expected for asphalt concrete that is 4 years old but this is probably a result of high initial AC penetration (approximately 200 penetration). These high-penetration values are desired for these climatic conditions to prevent pavement damage such as cracking caused by the environment.

Additional laboratory testing of the mixture indicates that the mix properties are generally satisfactory. The flow is higher than the maximum specified for airfield pavements which may be caused by excessive asphalt content or, possibly, the high amount of material passing the number 200 sieve. Properties such as stability, voids in the total mixture, and voids filled with asphalt are all within the suggested range for airfields. The asphalt mixture should provide good performance from environmental effects; however, a large volume of traffic may result in rutting due to the soft asphalt and high asphalt content.

The ALRS pavement at Hahn AB is in excellent condition. Distresses such as raveling were noted and indicate the need for a seal coat probably within the next 2 years. Surface drainage is a problem and should be corrected by ditching.

C. BASE COURSE EVALUATIONS

Since failure occurred in pavements tested at Wright-Patterson and Whiteman AFBs earlier than expected because the base course had not maintained the design strengths, a review was made of pavement evaluation reports for airfields in the Federal Republic of Germany (References 29-31) and the Republic of Korea (References 32-38). Only flexible pavements were considered. Data extracted from these reports are presented in Table A-25.

Most pavements found in the German evaluation reports contained a portland concrete cement layer. Those base course properties shown in Table A-25 are for sands (SP, SP-SM, SM, SC, and SM-SC) therefore would not be classified as a high-quality base course material. The inplace densities, where available for comparison, were all higher than the CE-55 densities, but for those with CBRs less than 80, the water content was higher than optimum from CE-55 compaction tests. Since the German materials are not classified as high-quality bases, it cannot be concluded that the base course CBR would be lower than 80. However, examples are shown where CBRs exist under 3-5-inch thick AC surfaces in the range of 20 to 50. Water contents in some cases are 1.2 to 4.6 percent above CE-55 optimum.

The Korean pavements are not as old as the German pavements and the bases have properties of a high-quality material. Water contents are lower than CE-55 optimum as well as inplace densities in some cases. Seven of the 11 base course strengths were less than 80 CBR. Of these seven base courses,

the densities of five were less than 100 percent of the CE-55 compaction test. The densities after construction are unavailable; therefore, it cannot be determined whether these low strengths are from construction or from the environment. These data do indicate a high probability that the inplace base course strengths will be less than 80 CBR. Furthermore, the importance of construction quality control, pertaining to grain size distribution and compaction requirements, is emphasized by these results.

Since ALRS pavements are not to be trafficked, no indication of loss of base course strength will be seen. In permanent airfield pavements surface distresses, such as alligator cracking or slight rutting, will indicate a structural problem which can be corrected with maintenance. In ALRS pavements, a structural evaluation must be conducted periodically to determine if loss in strength has occurred. Nondestructive testing offers a rapid, inexpensive means to structurally evaluate ALRS pavements. NDT data have been collected on ALRS type pavements and are presented below.

D. NONDESTRUCTIVE TESTING OF TRAFFIC TEST ITEMS

A falling weight deflectometer (FWD) was used to determine the pavement deflections before, during, and after traffic tests on the traffic test section and the pavements that were environmentally aged.

The FWD, which was used in this study, is a Model 8000 manufactured by Dynatest Consulting. A dynamic force is applied to the pavement surface by dropping a 440-pound weight onto a set of rubber cushions which results in an impulse loading. The applied force and pavement deflections are measured with load cells and velocity transducers. The drop weight can be varied from 0 to 15.7 inches to produce a force from 0 to 15,000 pounds. The load is transmitted to the pavement through an 11.8-inch diameter plate. The data acquisition equipment displays the resulting pressure in kilopascals and the maximum peak displacement in micrometers. As many as three displacement sensors may be recorded at one time.

FWD data collected were deflection basin measurements. Displacements were measured on the load plate and at distances of 12, 24, 36, and 48 inches from the center of the load plate. Because this particular model has only three transducers for deflection basin measurements, the five deflection points were obtained by dropping the weight twice and shifting the transducers to the additional spacings.

Data collected during testing of the three items of the test section are shown in Table A-26. Data from Wright-Patterson and Whiteman pavements are shown in Table A-27. Although detailed analysis of all the FWD data is not within the scope of this study, it should be recognized that the FWD was used to select the pavements tested at Wright-Patterson and Whiteman AFBs. The deflection measured at the center of the plate was correlated to passes to failure. Figure B-56 illustrates two relationships between passes to failure (3-inch rut depth) and plate deflection before trafficking. The reason for the two relationships is due to asphalt being stiffer in the aged pavements therefore giving less deflection. These relationships are simple indicators of pavement performance without the use of detailed analysis.

More detailed analytical methods can be employed for analysis of the pavement layers using NDT data.

One method is to model the pavement as a layered elastic system. FWD deflections can be used with a multilayered elastic program such as the Chevron program (Reference 39) to predict the modulus values. With the values and the known loading conditions for the F-4 aircraft, the strain at the top of the subgrade can be calculated and the total repetitions to failure can be estimated. Limiting strain criteria have been developed for airfield pavements (Reference 40) but do not include the low traffic passes that ALRS pavements are designed for. Using back-calculated modulus values in the limiting subgrade strain was calculated for all traffic test items. These are illustrated in Figure B-57. The single line extending through the points was extrapolated from the multiple lines reported in Reference 40. The greatest deviation from that line is the DBST pavements.

In summary, the FWD is an excellent device to predict the structural performance of ALRS pavements. Whether simple correlations or detailed analyses are used, a "go" or "no-go" can be determined for aircraft operations.

E. COST COMPARISONS

The cost of constructing a 50-foot by 5000-foot runway in Germany and Korea is compared for three pass levels and for condition of nonfrost and frost design. Unit in-place costs for construction in Korea are given in Table A-28. A conversion of 700 Korean won to 1 US dollar was selected. Unit in-place costs for construction in Germany are given in Table A-28. A conversion of 2.46 Deutsche marks to US dollar was used.

Cost comparisons for pavements for pass levels of 150; 50,000; and 300,000 for Germany and Korea are furnished in Table A-30. Air Force Manual 88-6, Chapter 1, "General Provisions for Airfield Design," specifies 300,000 passes of the specified lightweight aircraft (in this case the F-4) for conventional light load airfield pavements. It specifies 50,000 passes of the specified medium weight aircraft (for example, a C-141) for conventional medium load airfield pavements. Past AFESC cost comparisons, which indicated that the cost of the basic pavement structure of an ALRS were approximately one-third the cost of a "conventional" airfield pavement, used a portland cement concrete medium load airfield pavement in the comparison. This type of pavement is typical of those at air bases in USAFE (for example, Hahn AB). In the frost design procedure, the design for reduced subgrade strength controlled for the 150-pass level design. For the 50,000- and 300,000-pass level designs, the limited subgrade frost penetration design controlled.

Maintenance requirements for both permanent and ALRS pavements would be similar. An exception would be that the ALRS would require rolling with a multiwheel roller at least once a year after the spring thaw. Seal coats should be applied to the ALRS at 5 to 7 year intervals but probably are also needed on the permanent pavement. Permanent pavement maintenance would include painting, rubber removal, major rehabilitation (overlays, etc.)

and crack sealing (possibly in lieu of a seal coat). These requirements are related more to mission requirements (or change in mission requirements) and do not seem to apply in comparing life cycle costs. Therefore, the initial cost appears to be the controlling factor in a comparison of permanent to ALRS pavements.

SECTION VIII

CONCLUSIONS AND RECOMMENDATIONS

A. CONCLUSIONS

The following conclusions are presented:

1. Previous testing of thin asphaltic concrete (AC) surface treatment pavements had been conducted with tires with greater than 200-square inch contact areas or loads much less than expected for ALRS pavements. Results showed that the surface treatments or thin AC surface layers were capable of sustaining rolling wheel traffic but that skidding and turning were detrimental to surface treatments.

2. The minimum AC surface thickness requirements for an F-4 aircraft for 150 passes can be reduced from 3 inches to 2 inches.

3. A surface treatment will not support an F-4 when the pavement structure is designed for 150 passes.

4. A surface treatment subjected to environmental aging will break up under F-4 traffic and cause a foreign object damage potential for the aircraft engines.

5. For AC-surfaced pavements designed for minimum traffic levels, the base course is the critical component.

6. Aging index or degree of asphalt hardening due to aging was found to be inversely related to asphalt content by volume. In other words, the amount of aging decreases as the asphalt content increases.

7. The use of higher-penetration asphalt cement results in asphalt cement having a higher penetration after aging.

8. A change in aggregate gradation does not directly affect the laboratory-aging performance; however, a change in gradation usually requires a change in asphalt content which does affect the performance. A gradation should be selected (within limits) which requires the maximum asphalt content to be added to the mixture.

9. The reduced subgrade strength design procedure will provide the most economical design for the majority of ALRS pavements in Germany and Korea.

10. The base course strength of ALRS pavement will probably be less than 80 to 100 CBR after environmentally aging for 5-15 years.

11. Nondestructive pavement testing offers a rapid economical means of periodically determining a "go" or "no-go" for aircraft operations.

12. The cost of constructing AC-surfaced ALRS pavements designed for 150 passes of the F-4 aircraft will be approximately 60 percent of the cost of constructing permanent pavements designed for 300,000 passes of the F-4. This cost comparison is based upon costs for the basic pavement structure alone, assuming a level site. Not included are earthwork, drainage, utility relocation, NAVAID relocation, and other associated costs. When these costs are included, the percent difference in cost between ALRS and conventional pavements will be something less than 40 percent.

B. RECOMMENDATIONS

The following recommendations are presented for design, construction, and maintenance of ALRS pavements for 150 passes of an F-4 aircraft with a gross load of 60,000 pounds.

1. Design

The design procedures outlined in Department of the Navy, the Army and the Air Force, "Flexible Pavement Design for Airfields," (AFM 88-6, Chapter 2) (Reference 7) and Department of the Army and the Air Force, "Pavement Design for Seasonal Frost Conditions," (Draft AFM 88-6, Chapter 4) (Reference 21) should be followed. Exceptions and special requirements recommended are:

a. The minimum asphalt surface thicknesses be reduced to 2 inches.

b. The present requirements for airfield mixtures which require 3-5 percent voids in the total mixture should be changed to require 2-4 percent. The voids filled with asphalt requirements which now require 70-80 percent should be changed to 75-85 percent.

c. The present gradation requirements should be adopted for the ALRS asphalt mixtures. The requirement for the amount of material passing the number 200 sieve is 3-6 percent. It is desirable to maintain the amount of material passing the number 200 sieve on the low side of these requirements.

d. If two or more sources of asphalt cement are available, these asphalt cements should be tested in the laboratory to determine their aging characteristics prior to selecting source of asphalt cement.

e. The amount of voids in the in-place AC should not exceed 6 percent.

f. The upper 6-inch layer of base course should have a minimum 100 CBR and meet base course requirements specified in AFM 88-6, Chapter 2. The upper 4-inch layer must meet base course requirements and contain no more than 2 percent by weight passing the number 200 sieve for frost protection.

g. Adequate surface drainage be provided so as not to allow standing water near the ALRS pavement.

2. Construction

Strict quality control must be maintained throughout construction with particular emphasis on:

a. quality of the base course aggregate;

b. compliance with the 100 percent of CE-55 density requirements for the base course;

c. properties of the asphalt cement;

d. properties of the asphalt mix;

e. layer thicknesses.

3. Maintenance

Maintenance of the ALRS should be planned to include:

a. rolling the AC surface a minimum of once per year after the spring thaw with a 35-ton multiwheel roller or equivalent;

b. sealing the asphalt surface with a fog-seal coat every 5 to 7 years.

c. conducting nondestructive structural evaluations at 3-5 year intervals to ensure the pavement structural capacity has not deteriorated below the level for safe aircraft operations.

4. Additional Research

Conclusions and recommendations from this study are based on the design for a specific loading condition. If this loading is increased, or the contact pressure is increased, many of these recommendations will not apply. Therefore, additional research would be recommended for larger loads and higher tire pressures.

Additional accelerated aging tests based on the method described in Section V should be conducted on a broader range of aggregate types and asphalts. Field correlations should be made on laboratory test results. Test sections should be designed and constructed, based on laboratory work. Later, periodic testing of field mixes can be done to correlate field aging with laboratory aging.

Base course failure criteria should be established since high-quality materials are not available in all construction locations. Provisions should be developed to accommodate lesser quality base course materials.

A comprehensive field and laboratory study should be conducted to determine the extent and duration of thaw-weakened conditions. Results from such a study would provide information about the length of the severely

weakened pavement condition, i.e. a few days or a few weeks, and provide definitive estimates of the loss of a substantial strength. Results could provide a substantial reduction in thickness of ALRS pavements. The study could be conducted on in-service shoulder pavements.

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APPENDIX A

TABLES

TABLE A-1. LABORATORY JOB MIX FOR ASPHALTIC CONCRETE

US Standard Sieve Size	Percent Passing				
	Specified Limits	Job Mix Formula	Aggregate Coarse	Aggregate Fine	Aggregate Sand
			Percent to obtain mix		
			25	60	15
1 inch	---	---	---	---	---
3/4 inch	100	100	100	100	100
1/2 inch	82-96	97.8	91	100	100
3/8 inch	75-89	91.0	63	100	100
No. 4	59-73	73.2	17	91	96
No. 8	46-60	47.8	8	54	89
No. 16	34-48	33.0	6	32	82
No. 30	24-38	24.0	5	21	68
No. 50	15-27	11.2	4	15	8
No. 100	8-18	8.2	4	11	4
No. 200	3-6	6.3	3.4	8.6	2
Percent Bitumen		5.0			
Grade Bitumen		AC-20			
Stability (Marshall), lbs	1800 min	2240			
Flow, 0.01 inches	16 max	11			
Percent Voids Total Mix	3-5	3.9			
Percent Voids Filled	70-80	75.0			
Density - pcf		147.1			

TABLE A-2. SUMMARY OF CBR, DENSITY AND WATER CONTENT DATA FOR BASE COURSE AND SUBGRADE

	Pre-Traffic				After Traffic			
Item	Depth in.	Water Content, % Dry Weight	Density pcf	CBR	Depth in.	Water Content, % Dry Weight	Density pcf	CBR
				BASE COURSE				
1	2	2.2	144.9	91	2	1.9	148.0	110
2	1	2.7	144.4	85	1	2.2	148.8	107
3	1	2.3	144.1	96	1	2.1	145.3	103
				SUBGRADE				
1	12	26.2	93.4	6.3	12	26.5	92.8	7.0
	18	27.9	89.7	5.5	18	26.3	93.0	8.0
	24	26.6	92.9	6.9	24	25.9	93.3	6.0
	Avg	26.9	92.0	6.2	Avg	26.2	93.0	7.0
2	12	26.9	92.6	5.6	12	27.2	92.6	6.0
	18	27.7	91.2	7.0	18	27.5	91.9	6.7
	24	27.7	90.3	6.7	24	27.2	91.7	6.3
	Avg	27.4	90.4	6.4	Avg	27.3	91.1	6.3
3	12	26.8	92.1	6.7	12	26.7	91.9	7.3
	18	28.2	90.1	5.3	18	26.7	90.2	6.0
	24	27.7	91.0	5.6	24	28.7	90.5	5.0
	Avg	27.6	91.1	5.9	Avg	27.4	90.9	6.1

TABLE A-3. BITUMINOUS MIX ANALYSIS

US Standard Sieve Size	Percent Passing: Specified Limits	Percent Passing: From Extraction	Percent Passing: From Hot Bins
1 inch	---	---	---
3/4 inch	100	100	100
1/2 inch	82-96	95.7	97.8
3/8 inch	75-89	88.5	89.8
No. 4	59-73	71.7	71.9
No. 8	46-60	51.0	54.2
No. 16	34-48	35.5	39.1
No. 30	24-38	28.4	26.2
No. 50	15-27	10.0	7.9
No. 100	8-18	6.1	4.6
No. 200	3-6	4.3	3.7
Percent Bitumen		4.0	
Grade Bitumen		AC-20	
Stability (Marshall), lbs	1800 min	1694	
Flow, 0.01 inches	16 max	6	
Percent Voids Total Mix	3-5	4.9	
Percent Voids Filled	70-80	65.0	

TABLE A-4. TRAFFIC DATA FOR ALRS TEST SECTION

		Passes						
		Distributed Traffic				Channelized Traffic		
		Total Passes		Max. No. in One Path				
Item Number	Surface Type	1-in. rut	3-in. rut	1-in. rut	3-in. rut	1-in. rut	3-in. rut	Skids to Failure
1	2 in. AC	150	338	20	44	20	52	4
2	1 in. AC	120	150	14	20	24	41	2
3	DBST	12	48	2	6	9	29	1

TABLE A-5. AS-BUILT LAYER THICKNESSES

Item Number	Layer	Average Thickness Inch	Standard Deviation Inch
1	Asphalt	1.7	0.6
1	Base	8.2	0.6
2	Asphalt	1.4	0.3
2	Base	9.0	0.4
3	DBST	0.5	0.2
3	Base	9.4	0.5

TABLE A-6. PREDICTED TRAFFIC RESULTS

Item Number	Surface Type	Predicted Passes To 1-inch Rut Depth	Actual Passes To 1-inch Rut Depth
1	2 in. AC	113*	150
2	1 in. AC	144*	120
3	DBST	0**	12

* Subgrade CBR controlled.
** Base course CBR controlled.

TABLE A-7. PAVEMENT CONSTRUCTION HISTORY

Feature	Location	Pavement	Base Type	Base Thickness	Base In-place CBR	Subgrade Type	Subgrade In-place CBR	Constr. Date	Maintenance
					Wright-Patterson AFB, Ohio				
WP-1	Fire Equip. Parking Pad	3 in. AC	Gravel (GW)	6 in.	12	---	---	1961	---
WP-2	Shoulder Pavement T/W-17	1 in. AC over 2 in. AC	Clayey Gravel (GP-GC)	47 in.	33	---	---	1959	Overlay, 1971 Rejuvenator, 1979
WP-3	Apron D	2 in. AC	Gravel (GP)	12 in.	33	Clayey Sand (SC)	7	1974	Excavated 2-in. base MC-30 prime coat 2 in. AC, 1974
WP-4	Shoulder Pavement T/W-5	2 in. AC	Silty Gravel (GW-GM)	12 in.	72	Clayey Sand (SC)	8	1962	Rejuvenator, 1974
					Whiteman AFB, Missouri				
W-1	North Overrun R/W-01-09	1 in. DBST	Sandy Clayey Gravel (GC)	29 in.	33	---	---	1961	Seal coat, 1979
W-2	Shoulder Pavement T/W-9B	2.5 in. AC	(GC)	12 in.	102	Clay (CH)	4.2	1953	Slurry Seal, 1966
W-3	Blast Pavement Alert Apron	2.5 in. AC	(GC)	16 in.	37	Clay (CL)	4.2	1959	Slurry Seal, 1966

TABLE A-8. BASE COURSE AND SUBGRADE PROPERTIES AFTER TRAFFIC

Test Feature	Depth Inches	Material	CBR	Water Content	Dry Density - PCF In-Place (A)	Dry Density - PCF CE-55* (B)	Dry Density - PCF Water Content	Percent CE-55 Density
WP-1	3.0	Base (GW)	12	4.3	143.3	141.3	5.5	101
	9.0	Base (GW)	13	11.8	119.0	---	---	---
WP-2	3.0	Base (GP-GC)	33	5.4	145.3	143.1	5.2	102
	16.0	Base (GP-GC)	35	15.3	117.0	---	---	---
WP-3	2.0	Base (GP)	33	5.3	135.3	140.1	5.4	97
	14.0	Subg. (SC)	7	11.7	112.3	129.2	8.3	87
WP-4	2.0	Base (GW-GM)	72	3.6	138.7	143.3	5.7	97
	14.0	Subg.	8	20.6	100.8	---	---	---
W-1	1.0	Base (GC)	33	5.6	132.1	137.2	7.0	96
W-2	2.5	Base (GC)	102	4.5	140.3	137.5	6.3	102
	15.0	Subg. (CH)	4.2	24.1	97.4	120.1	12.8	81
W-3	2.5	Base (GC)	37	4.7	135.1	139.8	6.3	97
	19.0	Subg. (CL)	4.2	25.2	94.3	113.5	15.0	83

* Laboratory densities shown in this column are the CE-55 maximum densities at optimum water content.

TABLE A-9. BITUMINOUS MIX ANALYSIS FOR WRIGHT-PATTERSON AFB, OHIO

	Feature No.					
	WP-1	WP-2 (Top)	WP-2 (Bottom)	WP-3 (Top)	WP-3 (Bottom)	WP-4
US Standard Sieve Size						
1 inch	---	---	---	---	---	---
3/4 inch	100.0	100.0	100.0	---	---	100.0
1/2 inch	99.8	99.0	93.2	100.0	100.0	82.1
3/8 inch	96.2	92.2	84.2	94.6	93.5	70.1
No. 4	60.4	63.6	57.8	55.0	57.4	60.8
8	43.3	51.2	40.2	39.0	41.6	47.9
16	31.8	39.4	30.7	29.2	30.7	35.0
30	20.8	27.8	23.2	20.0	21.0	25.9
50	9.4	16.6	15.2	9.1	10.2	17.5
100	4.8	11.0	10.2	4.2	5.2	11.6
200	3.4	8.0	6.7	2.8	3.6	8.7
Percent Bitumen	5.0	6.6	4.0	5.2	5.3	4.4
Stability (Marshall) lbs.*	3235	3029	6087	2671	2698	3959
Flow - 0.01 inches*	12	22	12	12	13	13
% Voids - Total Mix*	5.7	1.0	6.5	3.7	3.8	4.8
% Voids - Filled*	66.3	94.0	58.2	76.7	76.5	68.0
Density - lbs/cu ft*	148.3	152.1	150.0	149.9	150.1	151.7
Field Den. - % of Lab. Den.	95.1	94.9	98.1	95.9	93.0	100.0
Field Core Thickness - in.	2.0	1.0	2.0	1.5	1.5	2.0
Agg. - Sp Gr	2.72	2.73	2.73	2.70	2.71	2.73
Agg. - % Water Absorption	1.0	0.8	0.8	0.9	0.9	0.9
Tests Performed on Recovered Asphalt						
Penetration	19	24	8	18	19	20
Viscosity-140^{o}F-Poises	43201	16565	329220	31562	28469	24662
Viscosity-225^{o}F-CST	10982	5694	51179	8337	8611	6282
Viscosity-275^{o}F-CST	1148	880	3727	1230	1251	1102

NOTE: Gyratory recompaction of field samples at 200 psi, 1^{o} angle & 30 revolutions.
* Recompacted data.

TABLE A-10. BITUMINOUS MIX ANALYSIS FOR WHITEMAN AFB, MISSOURI

	Feature No. W-1 (DBST)	W-2	W-3
US Standard Sieve Size			
1 inch	100.0	---	---
3/4 inch	96.0	100.0	100.0
1/2 inch	82.0	93.0	99.0
3/8 inch	62.0	84.0	91.0
No. 4	39.0	62.0	64.0
8	29.0	48.0	50.0
16	22.0	39.0	43.0
30	18.0	34.0	34.0
50	15.0	29.0	24.0
100	12.0	13.0	11.0
200	9.8	6.4	6.2
Percent Bitumen	5.6	4.8	4.6
Stability (Marshall) lbs.*	---	6136	3298
Flow - 0.01 inches*	---	12	12
% Voids - Total Mix*	---	6.9	3.1
% Voids - Filled*	---	60.6	77.9
Density - lbs/cu ft*	---	143.8	149.6
Field Den. - % of Lab. Den.	---	97.8	97.1
Field Core Thickness - in.	---	2.0	2.5
Agg. - Sp Gr	---	2.66	2.66
Agg. - % Water Absorption	---	0.9	0.6
Tests Performed on Recovered Asphalt			
Penetration	9	2	5
Viscosity-140°F-Poises	45291	513785	100809
Viscosity-225°F-CST	8092	35671	9530
Viscosity-275°F-CST	1060	3385	1170

NOTE: Gyratory recompaction of field samples at 200 psi, 1° angle and 30 revolutions.

* Recompacted data.

TABLE A-11. SUMMARY OF PAVEMENT DISTRESS DENSITIES - PERCENT

Feature	Pavement	Passes	Alligator L	Alligator M	Alligator H	Block L	Block M	Block H	Longit L	Longit M	Longit H	Raveling L	Raveling M	Raveling H	Rutting L	Rutting M	Rutting H	Remarks
Wright-Patterson AFB, Ohio																		
WP-1	3-in. AC	0				26			3									
		44*																Rut depth, 6 in.
WP-2	3-in. AC	0							8									
		337		15					9							5	6	
		481	5	16	30				8							20		
		643*	22		32							22			5	5	10	Rut depth,3.5 in
WP-3	2-in. AC	0	0.6									22						
		90*	22	32								22			5	5	10	Rut depth,3.5 in
WP-4	2-in. AC	0																
		119	33						8									
		162*	30	25	10												10	Rut depth,3.5 in
Whiteman AFB, Missouri																		
W-1	1-in.	0																
	DBST	100	40	38	22										10	10		Rut depth,1.75 in
		280*															15	Rut depth,3.0 in
W-2	2.5-in. AC	0		50						7								
		100	20	40	30												10	Rut depth,2.375 in
		132*	20	45	35												10	Rut depth,3.75 in
W-3	2.5-in. AC	0							0.7	0.8								
		50	40	30							2						10	Rut depth,2.25 in
		86*	30	25	20						2						10	Rut depth,3.5 in

NOTE: L, M and H denotes low, medium and high severity levels, respectively.
* Feature failed.

TABLE A-12. PREVIOUS ACCELERATED AGING TESTS ON ASPHALT MIXES

Reference	Aggregate	Asphalt	Aging Temp, °F	Aging Time	Strength Indicator	General Remarks
9	Ottawa sand	RC-4, MC-4 and various asphalts in 75-105 penetration range	325	1 to 8 hours	Compressive strength	Asphalt penetration of recovered asphalt was used to indicate degree of asphalt hardening/aging.
10	Sand	85/100 penetration from various sources	No data	30 months	Bearing strength	Asphalt penetration of recovered asphalt was used to indicate degree of asphalt hardening/aging.
11	Various mixes	Various asphalts in 60-300 penetration range	150	1 to 10 weeks	----	Field pavement specimens were aged in the laboratory. Asphalt penetration of recovered asphalt was used to indicate degree of asphalt hardening/aging. Up to 4 years of field aging data was also collected and observed.
12	Crushed stone, sand, and limestone dust	85/100 penetration (5 gradations)	140	12 & 63 days	Marshall stability	Air permeability of the mix was measured after accelerated aging. Asphalt penetration of recovered asphalt was used to indicate degree of asphalt hardening/aging.
13	Uniform and dense graded limestone	200/250 penetration	140	1 to 18 days	Compressive creep test	A nondestructive test method was used to determine mix strength and to indirectly determine asphalt hardening. Air was forced through specimens during accelerated aging.

TABLE A-13. AGGREGATE GRADATIONS AND FINENESS MODULUS VALUES

Sieve	Gradation, Percent Finer				
	A	B	C	D	E
3/4 inch	100	100	100	100	100
1/2 inch	89	89.4	79	100	91.4
3/8 inch	82	82.5	70	93	80.3
No. 4	65	65.4	52	80	65.4
No. 8	53	53.1	38	68	53.9
No. 16	40	40.5	29	53.5	40.3
No. 30	31	30.8	20	43.7	31.2
No. 50	21	21.3	13	32.0	20.8
No. 100	11	11.1	8.2	16.5	12.0
No. 200	4.4	5.6	2.8	8.2	5.5
	Fineness Modulus Values				
	3.97	3.95	4.70	3.13	3.96

TABLE A-14. PROPERTIES OF ASPHALTS

Property	ASTM Method	Test Results Southland	Texaco	Southland
Penetration at 77°F (25°C), 100 g, 5 sec, 0.1 mm	D 5	78	76	187
Specific gravity at 77°F (25°C)	D 70	1.028	1.029	1.023
Viscosity				
140°F (60°C), poises	D 2171	2118	2036	595
275°F (135°C), centistokes	D 2170	422	446	236
Flash Point, °F	D 92	540	595	465
Solubility, %	D 2042	99.95	99.8	99.9
Thin film oven test	D 1754			
Residual penetration at 77°F (25°C), 100 g, 5 sec, 0.1 mm	D 5	47	47	---
Viscosity at 140°F (60°C), poises	D 2171	9267	4997	1504
Viscosity at 275°F (135°C), centistokes	D 2170	---	658	---
Ductility at 77°F (25°C), cm	D 113	100+	---	150+

TABLE A-15. GENERAL AGING TEST RESULTS

Asphalt Penetration After Mixing	Age Days	Asphalt Penetration after Aging at Temperature 140°F	225°F	275°F
43	0	--	--	--
	1	43	31	21
	3	40	20	3
	7	39	16	--
	14	37	12	--
	28	25	--	--
49	0	--	--	--
	1	44	32	21
	3	41	25	12
	7	36	16	--
	14	35	9	--
	28	28	6	--

TABLE A-16. AGED AND UNAGED MIX PROPERTIES, STABILITIES, AND FLOWS

Mix	Mix Properties				Unaged		Aged	
	Asphalt Content, Percent Weight	Density, PCF	Air Voids, Percent	Voids Filled Percent	Stability, Pounds	Flow, 0.01 in.	Stability, Pounds	Flow 0.01 in.
LS	4.5	150.8	5.0	67.9	1563	13	----	--
Medium (B)		150.8	5.0	67.7	----	--	4456	12
S AC 20	5.0	152.3	3.3	78.0	1771	12	----	--
		151.7	3.7	76.1	----	--	3646	17
	5.5	153.2	2.0	86.5	1906	11	----	--
		152.2	2.6	82.8	----	--	3225	16
LS	4.5	151.4	4.5	70.0	1192	12	----	--
Coarse (C)		150.5	5.1	67.3	----	--	2652	21
S AC 20	5.0	152.2	3.3	78.4	1340	12	----	--
		151.5	3.8	75.9	----	--	2781	23
	5.5	152.8	2.2	85.8	1267	12	----	--
		151.9	2.8	82.5	----	--	2450	35
LS	6.0	148.[illegible]	4.1	77.2	1649	11	----	--
Fine (D)		148.8	4.3	76.5	----	--	3897	15
S AC 20	6.5	149.1	3.3	82.2	1744	10	----	--
		149.9	2.8	84.6	----	--	3286	13
	7.0	150.1	1.9	89.2	1772	14	----	--
		150.4	1.7	90.5	----	--	2490	12
LS	4.5	150.5	5.2	66.9	1305	9	----	--
Medium (B)		150.3	5.6	65.2	----	--	3868	18
T AC 20	5.0	152.4	3.3	78.2	1451	10	----	--
		151.4	4.2	73.9	----	--	3319	18
	5.5	153.2	1.9	86.7	1553	11	----	--
		152.3	2.9	81.7	----	--	2800	16
LS	4.5	151.4	4.6	69.9	994	9	----	--
Medium (B)		149.9	5.5	65.4	----	--	3375	11
S AC 5	5.0	152.2	3.3	78.1	1164	10	----	--
		151.1	4.0	74.4	----	--	2563	18
	5.5	152.8	2.3	85.4	1195	9	----	--
		152.5	2.4	84.4	----	--	2283	23
Gravel	7.0	141.0	6.0	72.1	1144	11	----	--
Medium (E)		141.1	5.9	72.2	----	--	2922	26
S AC 20	7.5	141.5	4.9	77.0	1414	12	----	--
		142.0	4.6	78.2	----	--	3120	20
	8.0	141.7	4.2	81.9	1514	12	----	--
		142.1	3.8	82.4	----	--	2670	16

TABLE A-17. ASPHALT MIX INDICES FOR 7-DAY AGING AT 225°F

Mix	Asphalt Content, Percent Volume	Asphalt Indices: Heating and Mixing (Avg)	Aging	Retained Penetration
LS	13.9	0.295	0.205	0.500
Fine (D)	15.2	0.295	0.103	0.602
S AC 20	16.4	0.295	-0-	0.705
LS	10.6	0.167	0.397	0.436
Coarse (C)	11.8	0.167	0.346	0.487
S AC 20	13.0	0.167	0.321	0.512
LS	10.5	0.141	0.410	0.449
Medium (B)	11.8	0.141	0.436	0.423
S AC 20	13.0	0.141	0.359	0.500
LS	10.5	0.026	0.513	0.461
Medium (B)	11.8	0.026	0.474	0.500
T AC 20	13.0	0.026	0.224	0.750
LS	10.5	0.128	0.465	0.407
Medium (B)	11.8	0.128	0.428	0.444
S AC 5	13.1	0.128	0.358	0.514
Gravel	15.4	0.103	0.385	0.512
Medium (E)	16.6	0.103	0.141	0.756
S AC 20	17.8	0.103	0.038	0.859

TABLE A-18. AIR FREEZING INDICES FOR SELECTED LOCATIONS

Location	Period of Record	Mean+	Coldest+	2nd Coldest+	3rd Coldest+	Design+
Whiteman AFB (Missouri)	1955-1980	289	724 ('77-'78)	649 ('76-'77)	428 ('61-'62)	686†
Pease AFB (New Hampshire)	1957-1978	590	817 ('76-'77)	794 ('67-'68)	776 ('60-'61)	806†
Wright-Patterson AFB (Ohio)	1947-1979	335	1058 ('77-'78)	886 ('76-'77)	733 ('62-'63)	892
Seneca Army Depot (New York)*	1958-1981	858	1327 ('62-'63)	1271 ('77-'78)	1210 ('76-'77)	1299†
Ft. Devens (Massachusetts)**	1949-1978	605	1277 ('77-'78)	1021 ('76-'77)	918 ('62-'63)	1072
Hahn AFB (Germany)	1953-1981	233	815 ('62-'63)	570 ('55-'56)	529 ('69-'70)	692†
Ramstein AB (Germany)	1952-1980	176	856 ('62-'63)	539 ('55-'56)	354 ('63-'64)	698†
Spangdahlem AB (Germany)	1953-1979	159	725 ('62-'63)	476 ('55-'56)	284 ('53-'54)	600†
Zweibrucken AB (Germany)	1966-1981	108	239 ('78-'79)	223 ('70-'71)	144 ('72-'73)	231†
Sembach AB (Germany)	1953-1968	235	853 ('62-'63)	519 ('55-'56)	362 ('63-'64)	686†
Bitberg AB (Germany)	1952-1981	159	678 ('62-'63)	473 ('55-'56)	310 ('78-'79)	487
Kusan AB (Korea)	1951-1981	160	545 ('76-'77)	410 ('62-'63)	300 ('80-'81)	418
Osan AB (Korea)	1953-1981	491	916 ('80-'81)	899 ('67-'68)	758 ('76-'77)	908†

*Used air temperature data from Aurora Research Farm which is located approximately 10 miles east of Seneca Army Depot.

**Used air temperature data from Fitchburg, Mass., which is located approximately 9 miles northwest of Ft. Devens.

+°F-days

†Based on average of two coldest years, other sites are based on the average of the three coldest years.

TABLE A-19. RESULTS FROM LABORATORY FROST-HEAVE TESTS

Material	Sample No.	Dry Unit Weight (pcf)	Saturation Percent	Average Rate of Heave (mm/day)	3-Day Max. Rate of Heave (mm/day)	Frost Classification
			Wright-Patterson AFB, Ohio			
WP-1	1	143.3	99	1.26	1.87	Low
(Base)	2	142.4	98	1.28	1.97	Low
WP-3	1	130.7	97	1.98	3.06	Low
(Subgrade)	2	130.0	97	1.57	1.90	Low
			Whiteman AFB, Missouri			
W-2	1	114.1	88	0.75	1.04	Very low
(Subgrade)	2	120.1	94	0.46	0.76	Negligible
W-3	1	115.0	92	0.48	0.90	Negligible
(Subgrade)	2	116.8	86	0.49	0.87	Negligible

NOTE: The standard frost heave test consists of a single cycle frost penetration. Multicycles would produce higher heave rates on the second and third cycles.

TABLE A-20. FIELD AND LABORATORY CALIFORNIA BEARING RATIO VALUES

Material	California Bearing Ratio, CBR		
	Field Tests*	Laboratory Unfrozen*	Laboratory After Freeze-Thaw
WP-1	11-12	244**	45.0
WP-3	10.5-15.0	48**	4.8
W-2	3.5-5.0	53**	---
W-3	1.8-6.8	50**	---

* Tests conducted by USAEWES.
** At optimum water content and density.

TABLE A-21. FROST-AREA SOIL SUPPORT INDICES FOR SUBGRADE SOILS FOR FLEXIBLE PAVEMENT DESIGN

Frost Group of the Subgrade Soil	F1 and S1	F2 and S2	F3 and F4
Frost Area Soil Support Index (FASSI)	9.0	6.5	3.5

TABLE A-22. REQUIRED THICKNESSES OF FLEXIBLE ALRS PAVEMENTS IN SEASONAL FROST AREAS

Frost Group of the Subgrade Soil	F1 or S1	F2 or S2	F3 or F4
Design Thickness, inches	8.5	10.5	14.5

NOTE: If the normal period CBR for the subgrade is less than the FASSI value listed in Table A-21, the normal period CBR value should be used in designing the pavement. In these instances, thicknesses would exceed those presented in Table A-22.

TABLE A-23. CONSTRUCTION MATERIAL SPECIFICATION FOR HAHN AB, ALRS

Binder Course 0/16 mm 1 1/2-inch (3.81-cm) thick

Aggregate size over 2 mm = 45-65% by weight
Aggregate size under 0.09 mm = 6-12% by weight
Bitumen B 200 = 5.4-6.7% by weight

Surface Course 0/8 mm 1 1/2-inch (3.81-cm) thick

Aggregate size over 2 mm = 40-60%
Aggregate size under 0.09 mm = 7-9%
Bitumen B 200 = 6.4-7.7%

AD-A145 800 DESIGN OF ALTERNATE LAUNCH AND RECOVERY SURFACES FOR ENVIRONMENTAL EFFECTS(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS GEOTE.. 2/3

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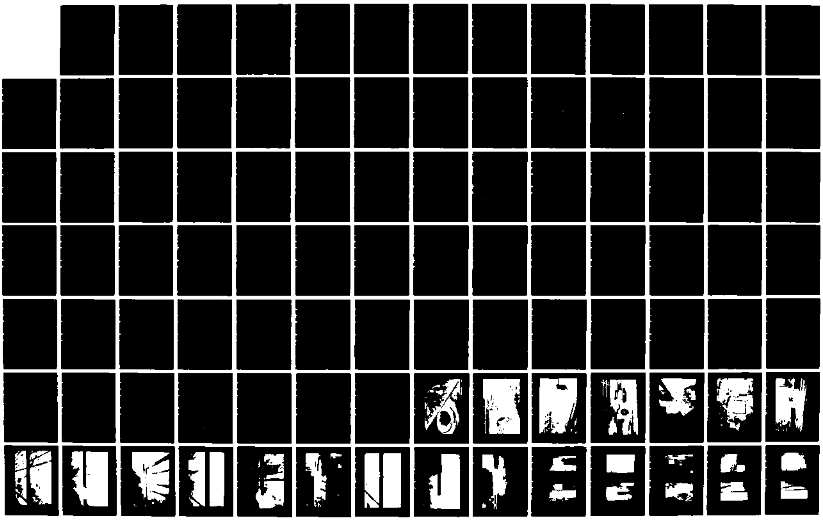

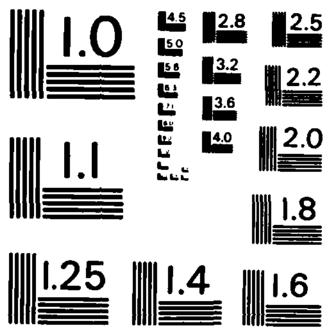

TABLE A-24. LABORATORY TEST RESULTS FOR HAHN AB, ALRS

US Standard Sieve Size	Surface Course	Binder Course
3/4 inch	100	100
1/2 inch	99.7	97.4
3/8 inch	98.5	85.4
No. 4	67.7	59.6
No. 8	51.8	45.2
No. 16	38.4	32.1
No. 30	28.1	22.8
No. 50	18.4	16.3
No. 100	12.4	13.1
No. 200	9.8	10.7
Percent Bitumen	6.6	5.1
Stability (Marshall) lbs *	2640	3633
Flow - 0.01 inches *	23	20
% Voids - Total Mix *	4.3	3.9
% Voids - Filled *	78.2	77.4
Density - lbs/cu ft *	151.8	165.3
Natural Sand, %	15.5	11.2
Agg. - Sp Gr	2.830	3.025
Agg. - % Water Absorption	1.5	1.5

Tests Performed on Recovered Asphalt

Penetration	64	104
Asphalt - Sp Gr	1.040	1.0425
Viscosity - 140^{o}F - Poisies	9010	2074
Viscosity - 225^{o}F - CST	5080	1812
Viscosity - 275^{o}F - CST	830	387

NOTE: Gyratory recompaction of field samples at 200 psi, 1^{o} angle & 30 revolutions.

* Recompacted data.

TABLE A-25. FLEXIBLE PAVEMENT BASE COURSE PROPERTIES FOR AIRFIELD PAVEMENTS IN GERMANY AND KOREA

						In-Place		CE-55			
Air Base	Feature No.	Location	AC Surface Thickness Inches	Base Course Classification	CBR Percent	Dry Density PCF	Water Content Percent	Dry Density PCF	Water Content Percent	Percent CE-55 Density	Construction Dates
					GERMANY						
Rhein-Main	A5B	Terminal Apron	3.5	SP-SM	35	115.0	11.1	95.0	9.8	100+	1949-1950 Overlay-1967
Rhein-Main	A13B	West Apron	4.5	SM	50	113.0	9.4	93.0	9.8	100+	1949-1950 Overlay-1965
Tempelhof	R4B	Inside Runway East End	5.0	Brick rubble SM-SC	90 20	102.0 110.0	7.8 12.0	--- 87.0	--- 7.4	--- 100+	1948 Overlay-1959
Tempelhof	R5B	Inside Runway East End	10.0	Stabilized sand SP-SM	100 15	--- ---	--- ---	--- ---	--- ---	--- ---	1964 Slurry seal-1970
Tempelhof	R7B	Outside Runway West End	12.0	SP	100	120.0	4.5	98.0	9.4	100+	1970
Tempelhof	R8C	Outside Runway Interior	12.0	Brick rubble SC	100 100	85.0 130.0	22.0 6.0	--- 102.0	--- 7.4	--- 100+	1948 construction Overlays in 1954, 1964, and 1971
Tempelhof	R10A	Outside Runway East End	9.5	Stabilized sand SP	100 100	--- 135.0	--- 6.0	--- ---	--- ---		1970
Leck	01B	West Overrun	11.0	SP-SM	17	---	---	---	---		----
Leck	02B	East Overrun	12.0	SP-SM	17	---	---	---	---		----
					KOREA						
R-222	R/W 18-36	Runway	3.25	GP	88	127.6	3.0	133.2	5.0	95.8	1979
R-113	R/W 14-32	Runway	3.0	GW-GM	32	138.9	3.0	136.7	5.5	101.6	1972
R-217	R/W 04-22	Runway	2.75	GP	64	128.8	4.0	134.3	6.1	95.9	1972
A-306	R/W 03-21	Runway	3.0	GP	129	138.4	3.0	134.3	6.3	103.1	1972
R-404	R/W 17-35	Runway	3.0	GW	59	128.5	5.0	132.5	5.3	97.0	1971
R-407	R/W 5-23	Runway	2.5	GW	104	140.3	3.0	139.2	5.4	100.8	1981
R-407	R/W 5-23	Runway	*	GP	38	127.8	3.0	135.5	5.2	94.3	1969
K-46	4	Apron	4.0	GP	54	137.8	2.0	134.5	6.1	102.5	1977
K-46	---	Parallel Taxiway	4.0	GP	105	146.0	3.0	134.5	6.1	108.5	1977
K-5	---	Parallel Taxiway	5.0	GP	49	125.7	4.0	133.7	6.3	94.0	1978
K-5	---	Parallel Taxiway	4.0	GP	60	130.6	3.0	133.7	6.3	97.7	1978

NOTE: * This base course was located beneath the original pavement constructed in 1969. In 1981, a base course with a 2.5-inch AC surface was placed over this original pavement. Properties from the new base course are given above.

TABLE A-26. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		ITEM 1 (0 passes)				
0+10	8,628	39.8	17.6	8.1	3.8	3.1
	14,099	65.0	31.4	14.3	6.5	5.2
0+20	8,546	43.5	20.7	9.8	4.6	3.4
	13,952	72.6	36.5	17.3	7.7	5.6
0+30	8,517	37.8	18.3	8.8	4.3	3.2
	13,999	62.2	31.5	18.5	7.4	5.9
0+40	8,466	42.7	21.6	10.2	4.5	3.5
	13,840	70.1	37.9		8.1	
		ITEM 1 (48 passes)				
0+10	8,358	53.5	23.6	8.9	3.9	3.0
	13,546	*	44.1	14.6	6.5	5.1
0+20	8,271	61.5	29.3	10.2	4.6	3.4
	13,305	*	45.1	17.7	7.3	5.6
0+30	8,239	56.2	25.6	9.8	4.6	3.5
	13,435	*	45.3	17.1	7.9	5.8
0+40	8,144	66.9	29.5	10.6	4.5	3.3
	13,197	*	51.0	18.9	7.8	5.7
		ITEM 1 (150 passes)				
0+10	8,326	55.5	31.1	10.4	4.7	3.2
	13,479	*	45.5	18.1	7.5	5.4
0+20	8,188	58.6	30.3	12.4	5.3	3.6
	13,273	*	51.4	21.9	8.8	6.1
0+30	8,136	62.5	30.3	11.8	5.2	3.7
	13,217	*	50.6	21.7	9.0	6.5
0+40	8,093	62.4	33.7	12.8	4.8	3.5
	13,141	*	56.9	22.8	8.7	5.8

* Deflection exceeded range of velocity transducer. (Sheet 1 of 3)

TABLE A-26. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		ITEM 1 (338 passes)				
0+10	8,180	54.5	24.2	11.1	5.4	3.9
	13,344	*	42.5	20.5	9.7	7.0
0+20	8,040	66.1	37.4	15.4	5.6	3.5
	13,077	*	74.6	26.6	11.6	5.8
0+30	8,112	54.5	34.3	12.2	5.8	3.9
	13,260	*	62.2	20.9	10.6	6.5
0+40	8,021	67.0	40.4	11.8	6.0	3.6
	13,046	*	63.2	22.0	10.0	6.2
		ITEM 2 (0 passes)				
0+65	8,342	56.1	28.0	10.6	5.5	3.9
	13,543	*	54.3	18.5	9.6	6.1
0+75	8,323	53.0	27.6	10.0	6.0	3.8
	13,575	*	51.0	19.3	10.4	6.7
0+85	8,252	55.6	31.5	10.6	5.8	3.6
	13,464	*	58.7	20.5	9.8	6.4
0+95	8,339	45.7	26.4	10.0	5.6	3.7
	13,734	75.9	45.9	18.3	9.5	6.2
		ITEM 2 (48 passes)				
0+65	8,048	*	30.1	10.4	4.8	3.6
	12,887	*	53.5	17.9	7.8	6.0
0+75	8,056	*	33.1	11.6	5.3	3.7
	12,915	*	55.3	19.7	8.7	6.2
0+85	8,053	*	33.5	11.8	5.3	3.9
	12,966	*	56.7	20.7	9.0	6.5
0+95	8,109	66.5	31.3	12.0	5.8	4.1
	13,213	*	53.3	22.8	9.8	7.0

(Sheet 2 of 3)

TABLE A-26. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONCLUDED)

Station No.	Force lbs.	Deflections Δ0-in. mils	Δ12-in. mils	Δ24-in. mils	Δ36-in. mils	Δ48-in. mils
		ITEM 2 (150 passes)				
0+65	8,017	73.8	32.3	13.0	5.6	4.1
	12,958	*	55.9	22.6	9.5	6.7
0+75	8,085	66.9	32.7	12.4	5.9	4.1
	13,058	*	53.5	22.2	10.0	7.0
0+85	8,088	60.6	34.4	13.0	6.4	4.1
	13,146	*	54.1	24.4	10.4	7.2
0+95	8,077	60.8	30.4	13.0	6.3	4.2
	13,213	*	53.1	23.6	10.7	7.3
		ITEM 3 (0 passes)				
1+20	8,167	65.7	30.3	13.0	6.7	4.1
	13,241	*	56.1	20.5	11.8	6.5
1+30	8,180	64.3	35.4	13.4	7.3	4.6
	13,340	*	51.8	23.2	11.8	7.7
1+40	8.164	57.7	27.2	12.2	6.3	4.3
	13,472	*	52.2	21.2	11.9	7.0
1+50	8,204	56.3	23.2	10.2	5.7	3.9
	13,638	*	45.2	18.1	10.4	6.3
		ITEM 3 (48 passes)				
1+20	7,902	*	29.1	13.8	5.5	3.8
	12,431	*	53.9	22.8	9.1	6.2
1+30	7,842	*	23.6	13.2	5.4	4.2
	12,224	*	41.3	19.1	8.0	6.3
1+40	6,141	*	22.0	11.4	5.4	3.8
	9,610	*	37.8	20.1	8.5	6.1
1+50	8,005	*	23.4	11.0	5.6	4.1
	12,756	*	43.1	19.9	9.0	6.3

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
			TRAFFIC TESTS			
			WP-1 (0 passes)			
0+05	8,803	62.8	18.9	3.9	1.3	2.3
	13,205	*	29.9	2.4	1.4	2.8
0+15	8,819	43.7	17.7	3.2	1.6	1.2
	13,236	60.4	28.0	5.0	1.7	2.4
0+25	8,851	47.4	23.5	3.5	1.9	1.1
	13,352	66.3	34.8	5.3	3.0	1.7
			WP-2 (0 passes)			
0+05	8,994	20.9	9.4	3.2	1.5	1.5
	13,538	27.8	13.5	4.3	2.3	1.8
0+15	8,898	24.2	12.2	3.4	1.9	1.5
	13,538	31.3	17.6	4.7	2.7	2.1
0+25	8,867	23.2	11.3	3.5	2.1	1.5
	13,522	31.5	16.1	4.8	3.0	2.3
			WP-2 (337 passes)			
0+05	8,612	*	42.9	10.6	1.5	1.7
0+15	8,596	*	62.6	9.1	1.0	1.8
	13,093	*	65.7	10.6	1.5	2.7
0+25	8,724	*	51.2	5.9	1.5	1.6
	13,363	*	50.4	7.9	2.2	2.5
			WP-2 (481 passes)			
0+05	9,375	*	49.2	12.6	3.6	2.2
	13,888	*	52.4	14.6	3.5	2.5
0+15	9,296	*	58.7	11.4	2.4	2.0
	13,761	*	63.4	12.6	3.0	2.9
0+25	9,200	*	41.7	5.5	2.2	3.4
	13,650	*	46.9	7.1	3.2	3.3

* Deflection exceeded range of velocity transducer. (Sheet 1 of 8)

TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		TRAFFIC TESTS (CONTINUED)				
		WP-2 (643 passes)				
0+05	8,787	67.7	35.4	9.8	2.4	1.6
	13,379	*	42.1	11.0	2.8	2.8
0+15	8,771	*	46.9	6.3	1.2	1.6
	13,284	*	49.2	8.7	2.0	2.4
0+25	8,708	*	31.1	4.7	1.6	2.8
	13,205	*	38.2	5.1	3.5	2.8
		WP-3 (0 passes)				
0+05	9,200	45.7	23.6	6.3	2.5	2.4
	13,618	66.3	36.2	8.3	3.1	2.7
0+15	9,200	44.5	21.6	4.9	2.2	2.0
	13,665	63.3	33.9	7.7	2.7	2.5
0+25	9,184	55.7	28.0	4.3	2.6	2.6
	13,602	77.2	40.6	6.7	3.3	2.7
		WP-3 (90 passes)				
0+05	8,464	*	62.6	16.1	2.4	1.6
	12,172	*	97.6	24.8	2.3	1.0
0+15	8,168	*	77.2	21.7	4.4	2.4
	11,854	*	*	31.5	5.6	3.0
0+25	7,786	*	*	21.7	7.0	2.6
	11,314	*	*	31.5	12.2	3.0

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
			TRAFFIC TESTS (CONTINUED)			
			WP-4 (0 passes)			
0+05	9,137	37.2	19.3	5.4	1.9	1.2
	13,427	52.4	28.8	8.5	2.6	1.8
0+15	9,121	32.1	14.3	3.9	1.5	1.3
	13,570	44.3	22.5	6.2	2.1	2.2
0+25	9,057	32.3	14.3	3.7	1.5	1.4
	13,475	44.5	22.6	5.9	2.0	1.3
			WP-4 (162 passes)			
0+05	8,295	*	*	13.0	2.6	1.7
	11,965	*	*	20.0	4.5	2.8
0+15	8,692	*	58.0	7.9	3.1	1.3
	12,648	*	*	13.4	4.1	3.4
0+25	8,279	*	*	7.5	2.3	1.3
	11,886	*	*	11.0	4.8	2.3
			TURNING TESTS (0 passes)			
			WP-1			
---	4,052	27.0	13.8	3.1	0.6	0.6
	9,200	60.2	31.4	6.8	1.0	1.3
	13,634	*	39.3	9.1	0.8	2.0
			WP-2			
---	3,893	11.8	5.2	1.4	0.8	0.6
	9,184	22.5	10.6	3.2	1.9	1.4
	13,793	30.7	14.7	4.8	2.9	2.1

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		TURNING TESTS (0 passes) CONTINUED				
		WP-3				
---	4,004	16.9	8.8	2.8	1.2	0.8
	9,200	38.8	21.2	7.2	2.8	1.9
	13,602	56.3	31.9	10.7	4.2	2.8
		WP-4				
---	4,243	14.9	7.2	2.3	0.7	0.7
	9,296	32.7	17.7	5.4	1.7	1.5
	13,808	46.5	27.0	7.9	2.4	1.7
		TRAFFIC TESTS				
		W-1 (0 passes)				
0+05	9,081	36.5	8.5	5.1	3.7	2.9
	14,063	53.4	11.6	7.8	5.5	4.3
0+15	9,049	43.1	9.9	4.9	3.7	2.9
	13,955	59.7	15.6	7.5	5.6	4.3
0+25	9,033	35.8	7.1	5.2	3.8	2.8
	14,019	51.3	11.5	7.8	5.9	4.4
		W-1 (50 passes)				
0+05	9,101	48.9	22.1	8.8	5.0	3.6
	13,982	69.5	32.6	13.5	7.5	5.4
0+15	8,930	62.1	25.2	8.2	5.4	3.5
	13,781	*	37.8	13.0	7.7	5.4
0+25	8,890	60.3	23.6	8.1	5.1	3.6
	13,721	*	35.2	13.0	8.1	5.4

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	$\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		Deflections				
TRAFFIC TESTS (CONTINUED)						
W-1 (100 passes)						
0+05	8,941	76.5	33.4	11.5	6.7	3.6
	13,693	*	49.2	19.0	9.0	5.6
0+15	8,771	*	44.9	5.9	5.6	3.8
	13,518	*	55.3	9.5	9.1	6.0
0+25	8,815	*	36.4	11.5	5.9	3.8
	13,448	*	52.0	18.5	9.5	6.1
W-1 (150 passes)						
0+05	8,644	67.1	33.4	11.4	5.3	3.7
	13,371	*	49.4	19.7	8.8	5.7
0+15	6,491	*	33.5	11.6	5.7	3.2
	10,333	*	53.0	18.9	9.1	5.3
0+25	8,263	*	36.2	15.0	7.1	3.9
	13,066	*	56.3	20.2	11.7	6.3
W-1 (200 passes)						
0+05	9,200	*	48.2	17.9	6.5	3.7
	13,999	*	68.5	26.0	9.4	5.9
0+15	8,871	*	60.6	17.9	7.3	3.7
	13,594	*	*	26.8	9.7	5.7
0+25	8,673	*	70.1	28.4	11.8	4.1
	13,400	*	*	42.3	18.4	5.9

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		TRAFFIC TESTS (CONTINUED)				
		W-1 (280 passes)				
0+05	4,151	71.3	26.4	8.3	2.8	2.2
	9,176	*	50.6	16.6	6.9	5.0
0+15	3,432	*	37.1	8.7	2.6	1.5
	7,624	*	68.5	19.8	6.7	3.9
0+25	4,020	*	38.3	11.3	4.8	3.0
	8,673	*	*	26.3	11.2	6.0
		W-2 (0 passes)				
0+05	9,160	48.2	28.4	11.0	5.9	4.6
	14,173	67.0	41.9	17.2	9.7	7.3
0+15	9,137	44.5	27.5	11.3	5.9	4.5
	14,106	62.8	41.3	17.5	9.8	7.2
0+25	9,149	45.0	25.9	9.7	5.3	3.8
	14,118	62.8	39.2	15.7	8.7	6.4
		W-2 (50 passes)				
0+05	4,080	61.2	35.4	10.4	4.4	2.8
	8,390	*	76.4	20.9	9.9	7.2
0+15	4,028	52.7	31.9	9.3	5.0	2.5
	8,446	*	72.6	19.1	10.1	6.1
0+25	4,000	54.8	32.5	8.6	5.6	2.3
	8,390	*	70.3	33.5	9.6	5.7

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONTINUED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		TRAFFIC TESTS (CONTINUED)				
		W-2 (100 passes)				
0+05	3,583	*	58.7	12.3	4.6	3.3
0+15	3,899	68.0	30.5	11.6	5.1	3.0
0+25	3,822	60.3	32.9	9.7	4.2	2.4
		W-3 (0 passes)				
0+05	8,827	46.1	17.0	7.8	4.2	3.2
	13,844	68.9	27.2	12.2	6.5	4.9
0+15	8,934	41.8	20.1	7.9	4.4	3.0
	13,884	60.9	30.9	12.2	6.8	4.9
0+25	8,875	42.2	18.4	7.1	4.1	2.8
	13,848	61.4	28.6	10.9	6.4	4.7
		W-3 (50 passes)				
0+05	4,044	*	41.9	10.0	3.3	2.2
0+15	4,020	66.0	41.9	10.9	3.6	2.2
	8,267	*	*	24.2	6.4	4.2
0+25	3,958	*	51.2	10.7	3.0	1.8
	8,064	*	*	20.0	4.5	2.2
		W-3 (86 passes)				
0+05	(Unable to use Station 0+05)					
0+15	4,004	*	47.2	10.1	2.6	2.0
0+25	2,300	*	45.1	8.2	2.3	2.2

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TABLE A-27. FALLING-WEIGHT DEFLECTOMETER DEFLECTION BASIN DATA (CONCLUDED)

Station No.	Force lbs.	Deflections $\Delta_{0\text{-in.}}$ mils	$\Delta_{12\text{-in.}}$ mils	$\Delta_{24\text{-in.}}$ mils	$\Delta_{36\text{-in.}}$ mils	$\Delta_{48\text{-in.}}$ mils
		TURNING TESTS (0 passes)				
		W-1				
---	4,481	24.9	7.0	3.9	2.4	2.0
	9,864	49.4	16.7	8.8	5.5	3.7
	14,305	62.5	23.7	12.4	8.1	5.3
		W-2				
---	4,319	23.4	13.9	7.0	3.7	3.1
	9,057	46.2	29.7	15.0	7.7	5.9
		W-3				
---	4,390	21.6	9.4	3.3	2.1	1.5
	9,121	42.4	21.5	7.1	4.3	3.1
	14,055	65.9	35.6	11.4	6.6	4.9

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TABLE A-28. COST ANALYSIS DATA (KOREA)

	Costs (1982)	
	Korean Won	US Dollar
Subgrade Preparation, (sq. yd.)	240	0.34
Base Course Material, (cu. yd.)*	8,500	12.14
Subbase Course Material, (cu. yd.)*	6,800	9.71
Placement of Base or Subbase, (cu. yd.)	2,600	3.71
Rough Grading, (sq. yd.)	320	0.46
Fine Grading, (sq. yd.)	150	0.21
Tack Coat, (sq. yd.)	440	0.63
Prime Coat, (sq. yd.)	510	0.73
Asphaltic Concrete, (ton)**	29,800	42.57
Placing Asphaltic Concrete, (ton)	5,800	8.29

* Includes hauling.
**Does not include hauling.

TABLE A-29. COST ANALYSIS DATA (GERMANY)

	Costs (1981)	
	Deutsch Mark	US Dollar
Subgrade Preparation, (m^2)		
Finish grading and compaction	1.80	0.73
Compaction only	1.25	0.51
Subbase and Base Course, (m^3)		
Nonfrost susceptible subbase	35.00	14.23
Graded (0-32 mm) NFS subbase	50.00	20.33
Hydraulically bound gravel base	55.00	22.38
Hydraulically bound crushed rock	65.00	26.42
Graded (0-32 mm) gravel base	55.00	22.38
Graded (0-32 mm) crushed rock	55.00	22.38
Bituminous base	225.00	91.46
Asphaltic Concrete, (m^3)		
Surface course	300.00	121.95
Binder (intermediate) course	250.00	101.63
Surface Treatments, (m^2)		
Single surface treatment	4.00	1.63
Lime and Cement Stabilized Soils, (m^2)		
Lime treatment, 3%, 20 cm depth	5.00	2.03
Cement treatment, 6%, 15 cm depth	5.00	2.03

NOTE: The hydraulically bound base courses are plant mixed, using either cement or lime. The stabilized soils are mixed in place. All the above amounts are for the cost in place.

TABLE A-30. ALRS COST COMPARISONS

Pass Level	Pavement Structure	1981 Cost - $	Location	Remarks
		NONFROST DESIGN		
150	2-inch AC/8-inch base	266,367	Germany	Seal coat after 5-7 years; approx. cost-$20,278.
50,000	3-inch AC/6-inch base/ 12-inch subbase	412,664	Germany	
300,000	2-inch AC/10-inch base/ 10-inch subbase	448,627	Germany	
150	2-inch AC/6-inch base	280,696	Korea	
50,000	3-inch AC/9.5-inch base	400,023	Korea	
300,000	3-inch AC/12-inch base	430,626	Korea	
		FROST DESIGN		
150	2-inch AC/6-inch base/ 6-inch subbase	290,354	Germany	Seal coat after 5-7 years; approx. cost-$20,278.
50,000	3-inch AC/6-inch base/ 19-inch subbase	496,618	Germany	
300,000	3-inch AC/6-inch base/ 19-inch subbase	496,618	Germany	
150	2-inch AC/6-inch base/ 6-inch subbase	355,649	Korea	
50,000	3-inch AC/6-inch base/ 19-inch subbase	579,636	Korea	
300,000	3-inch AC/6-inch base/ 19-inch subbase	579,636	Korea	

NOTE: F-4 aircraft
Design Load = 60 kips
Design CBR = 7 (Germany); CBR = 15 (Korea)
Runway Size = 50 feet x 5,000 feet

APPENDIX B

FIGURES

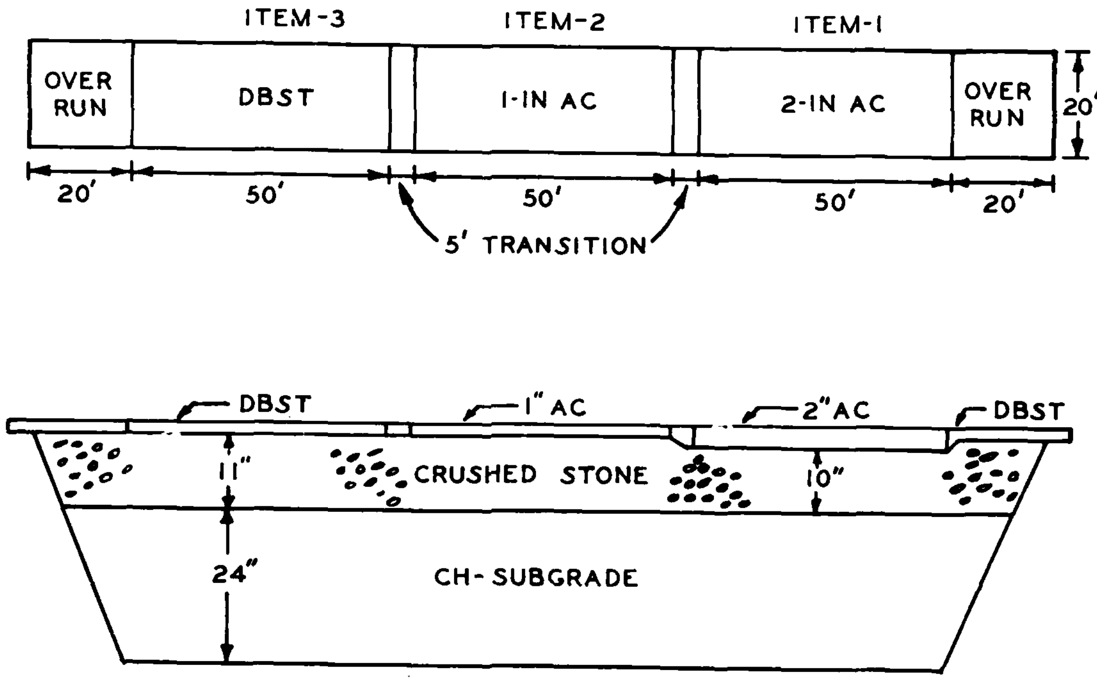


Figure B-1. Plan View and Layout of Traffic Test Section

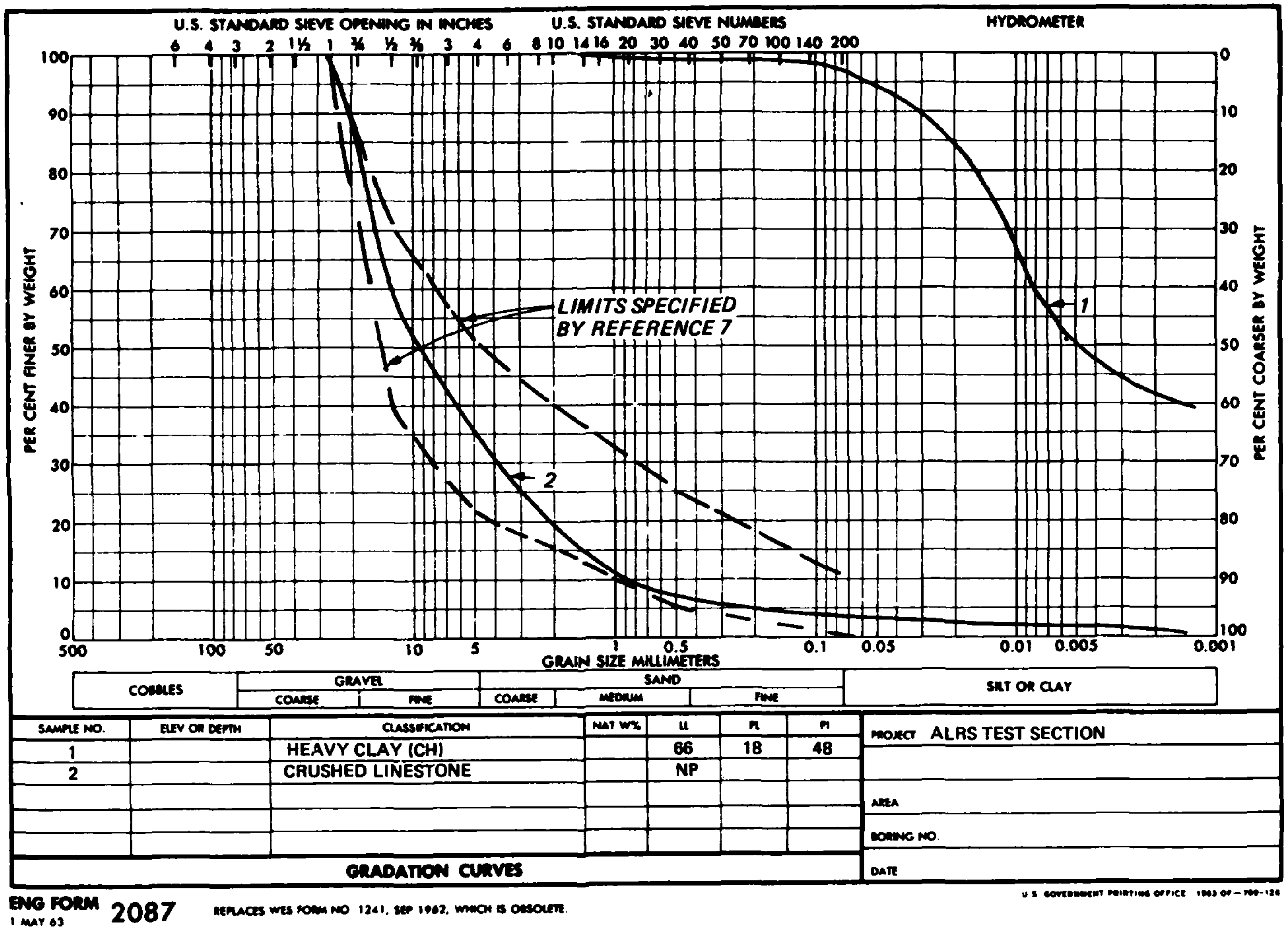


Figure B-2. Gradations for Base Course and Subgrade Material

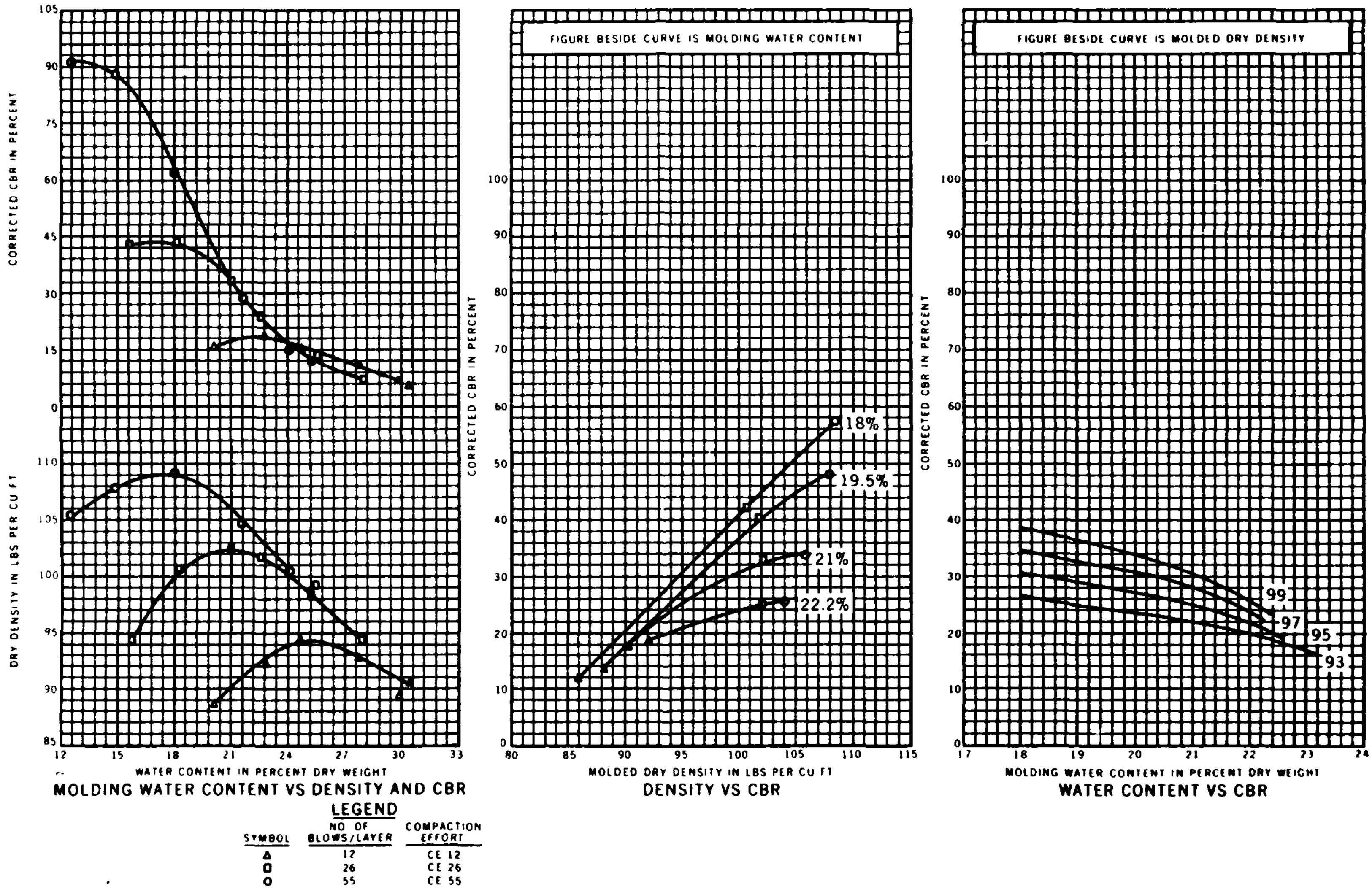


Figure B-3. Laboratory Compaction and CBRs for CH Subgrade (Unsoaked)

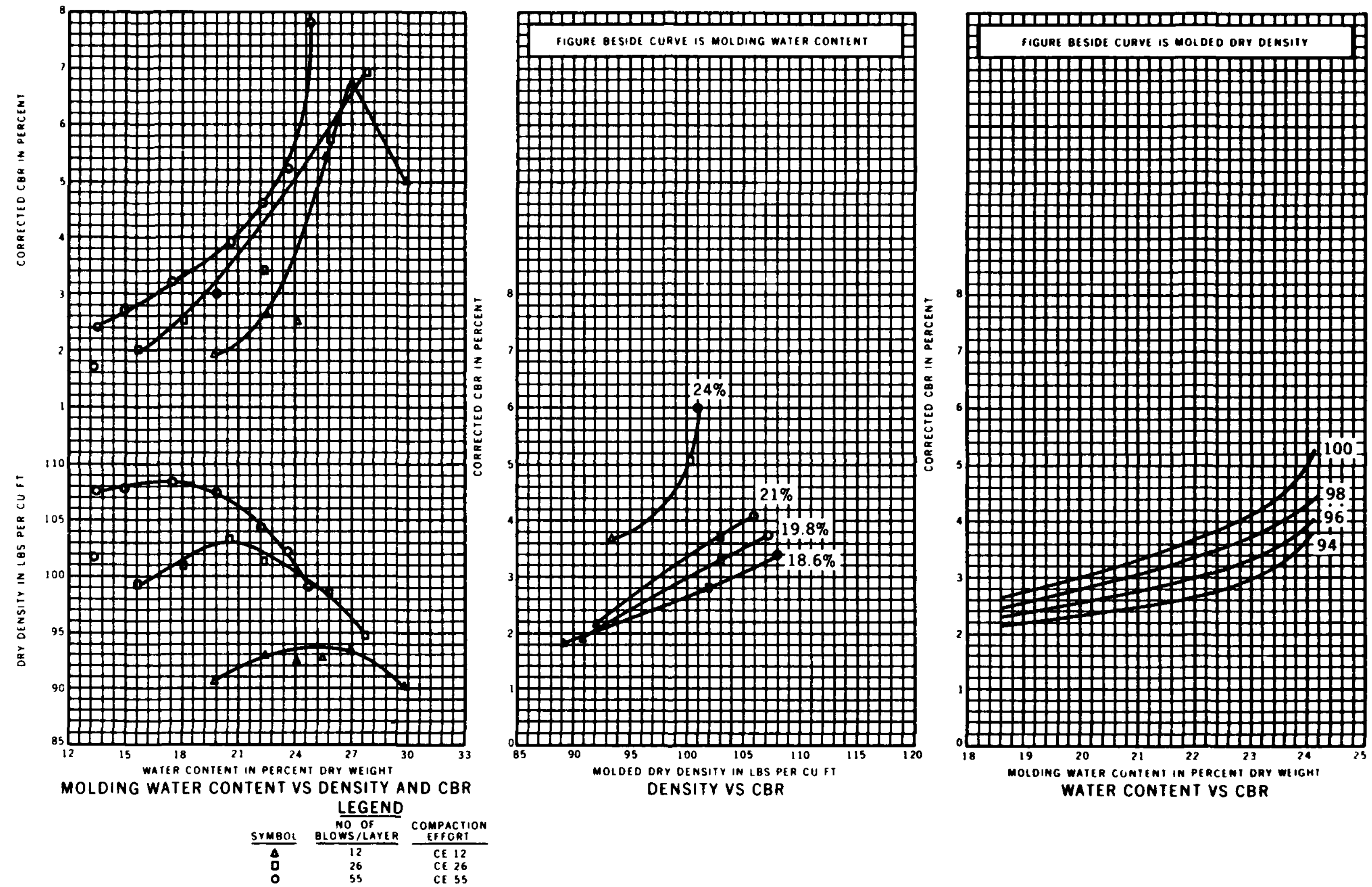


Figure B-4. Laboratory Compaction and CBRs for CH Subgrade (Soaked)

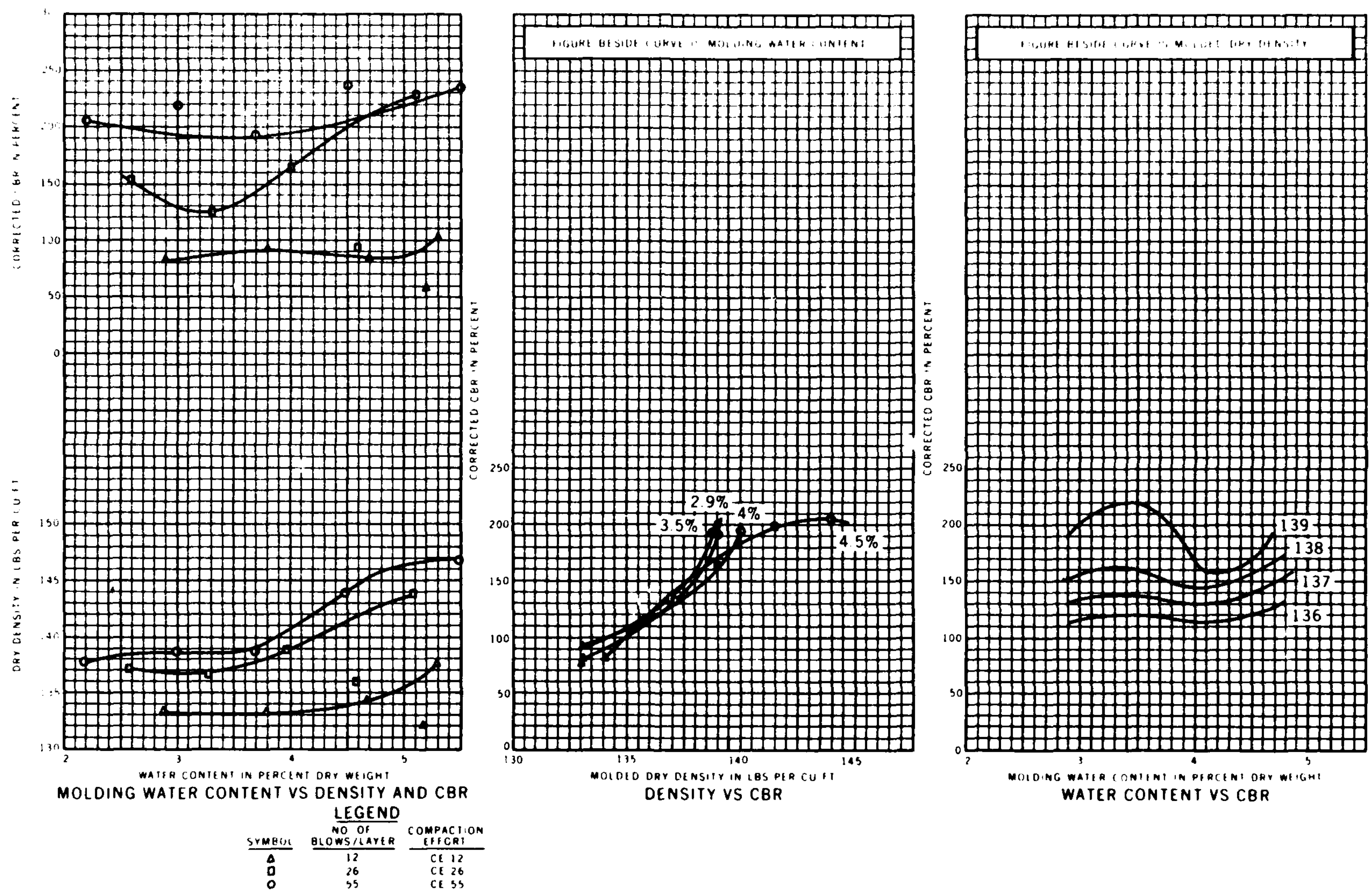


Figure B-5. Laboratory Compaction and CBRs for Base Course (Unsoaked)

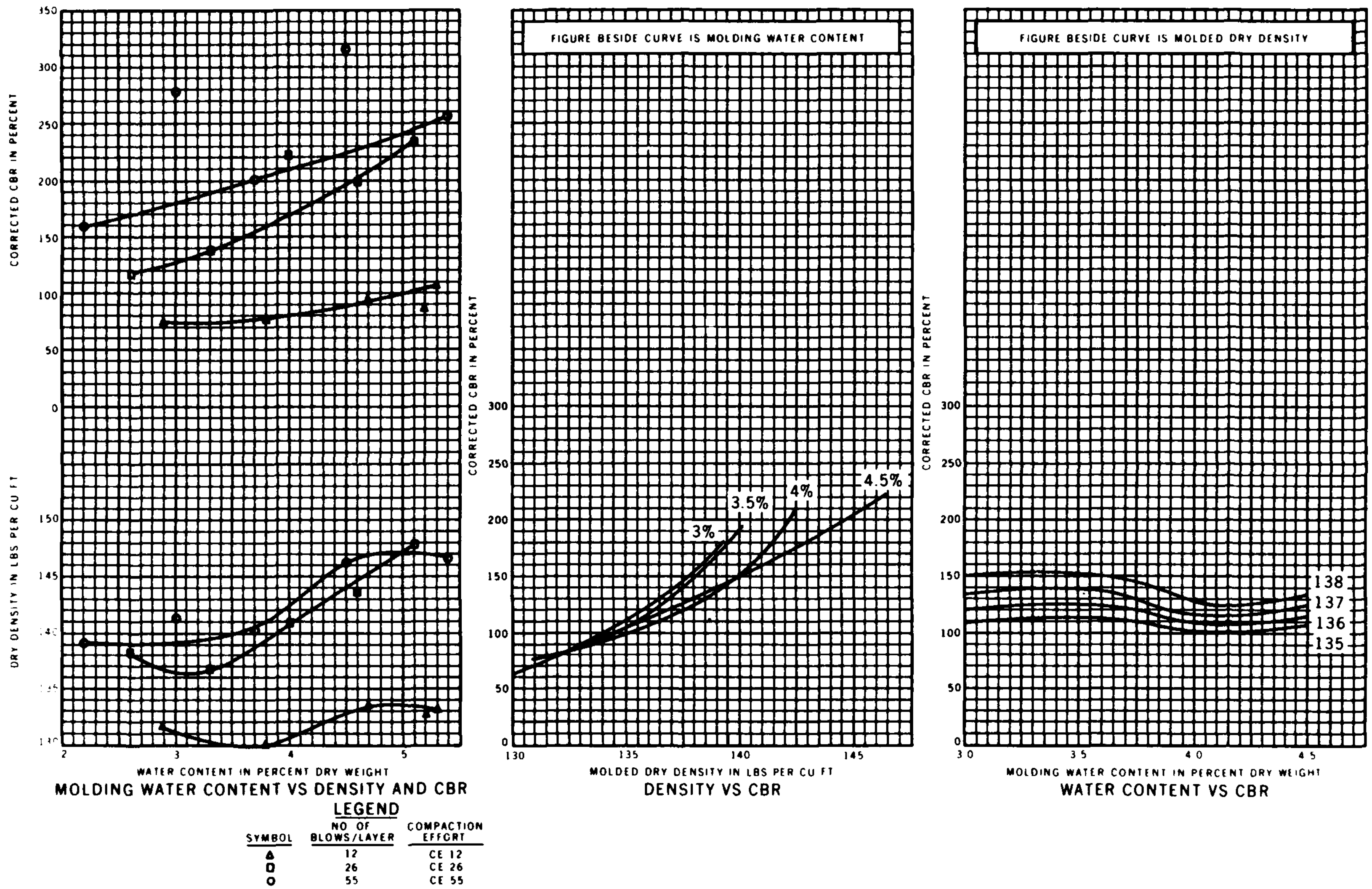


Figure B-6. Laboratory Compaction and CBRs for Base Course (Soaked)

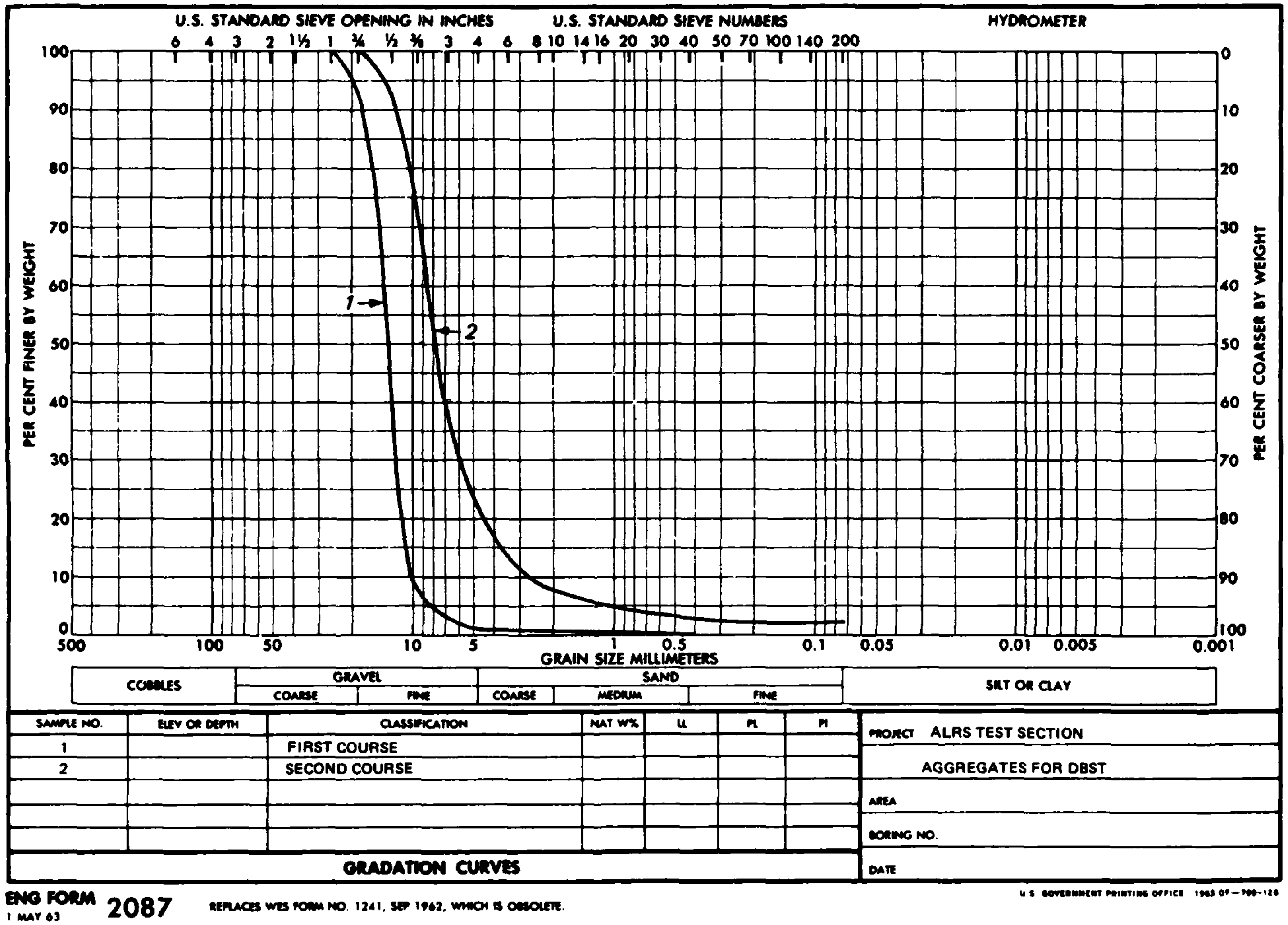


Figure B-7. Gradations for Double-Bituminous Surface Treatment Aggregates

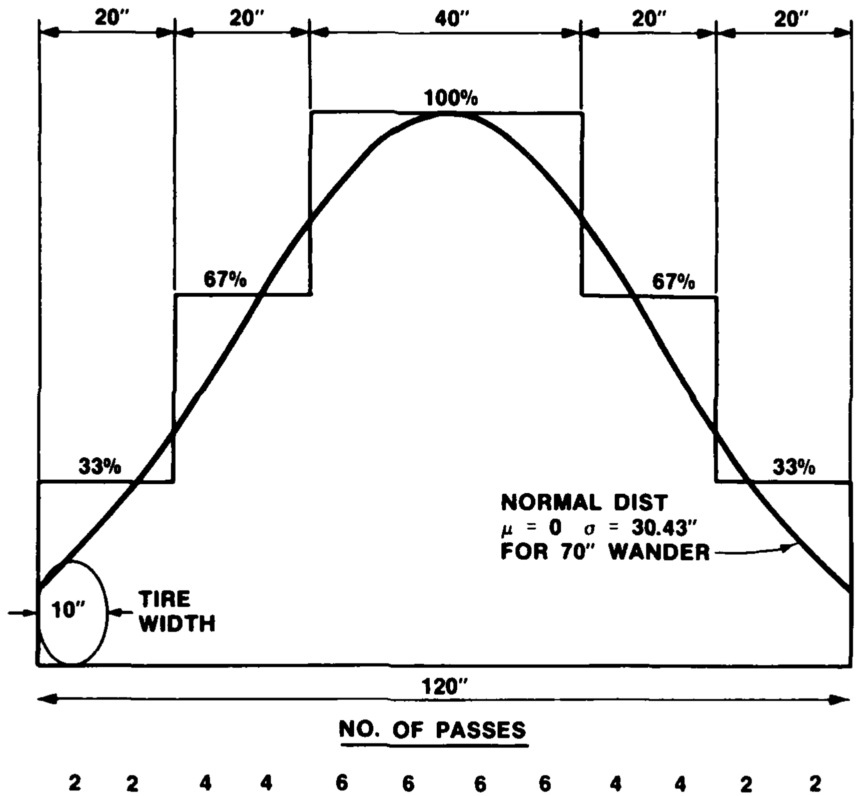


Figure B-8. Traffic Distribution Pattern for F-4 Traffic Tests

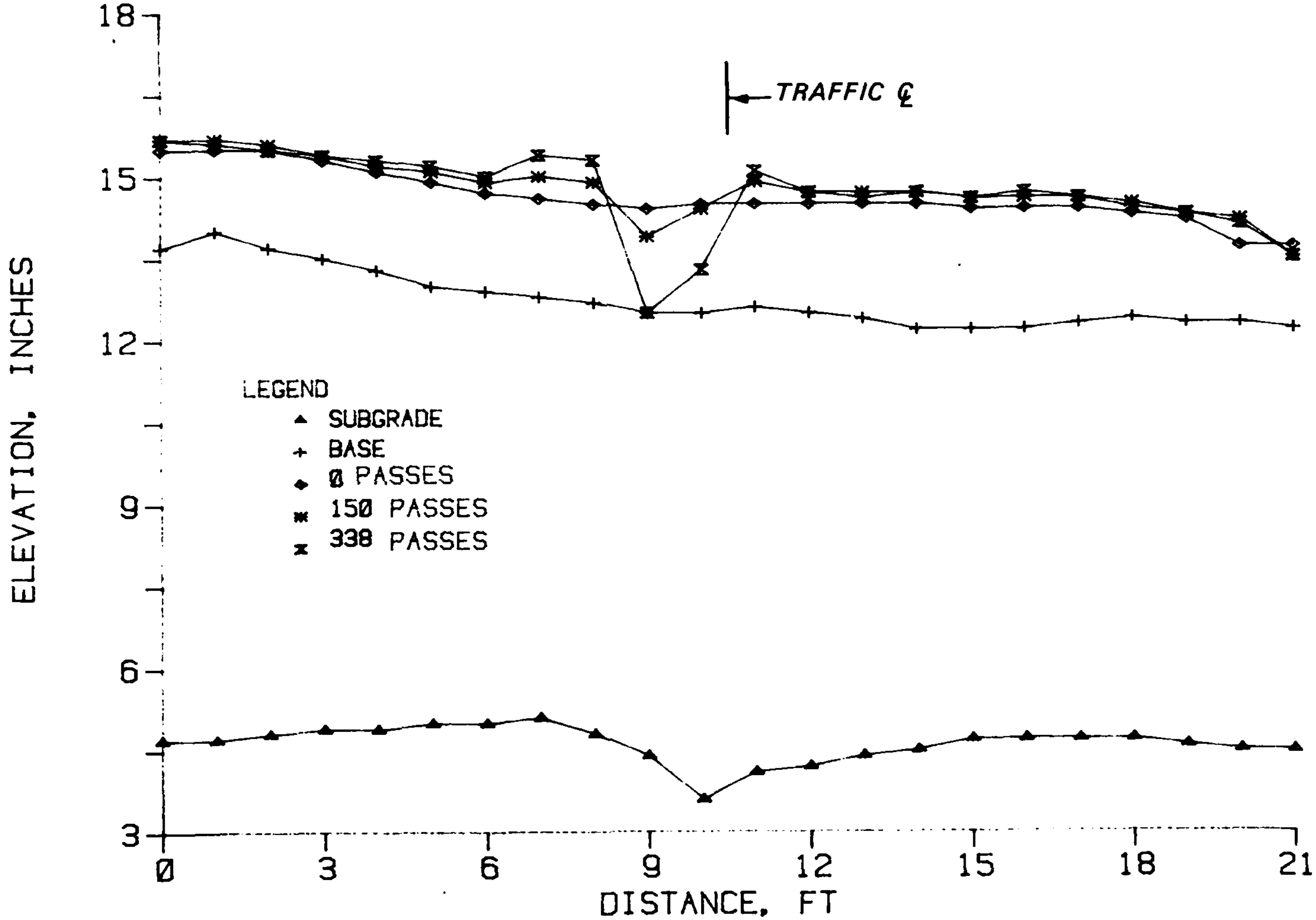


Figure B-9. Cross Section of Item 1

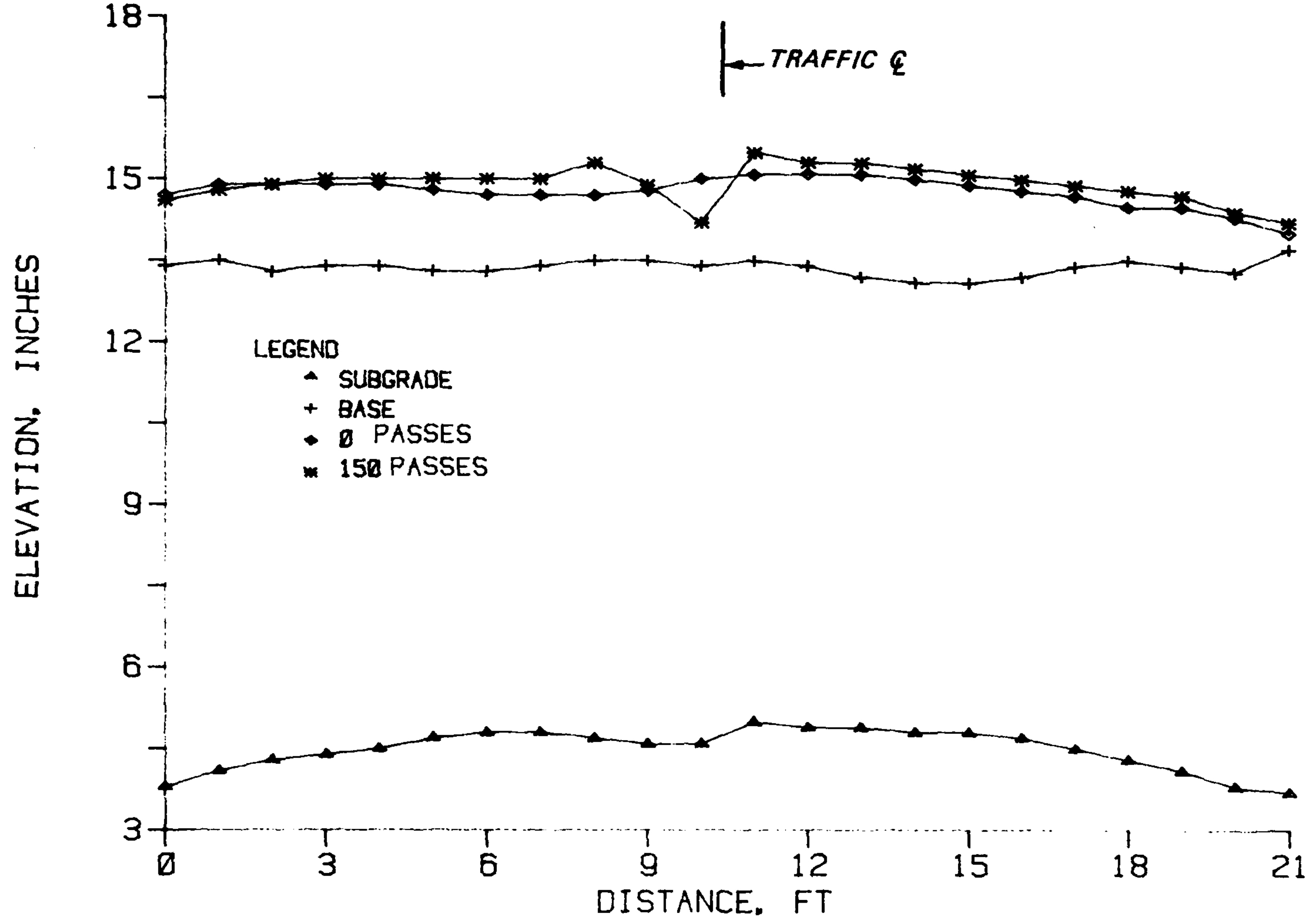


Figure B-10. Cross Section of Item 2

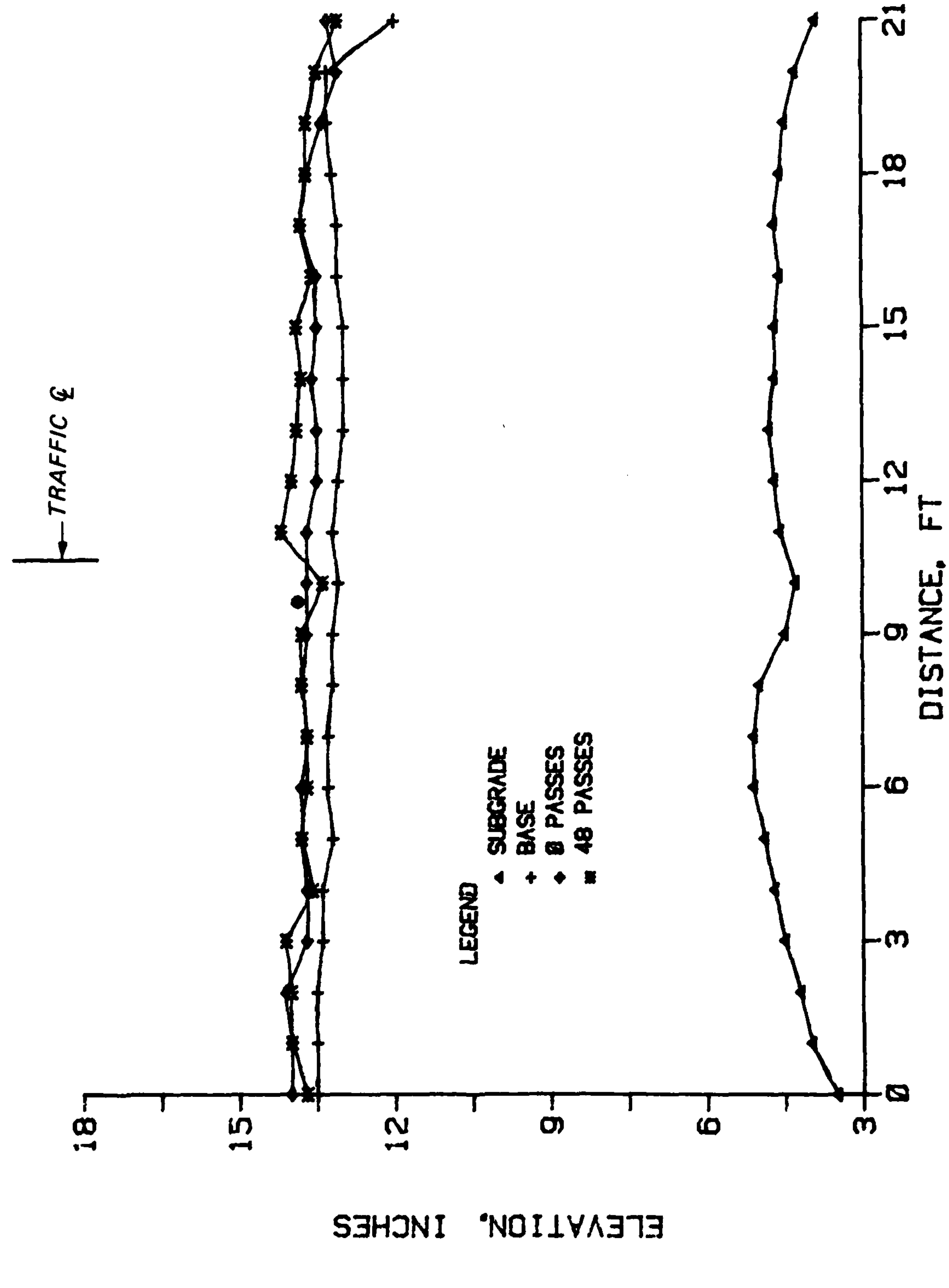


Figure B-11. Cross Section of Item 3

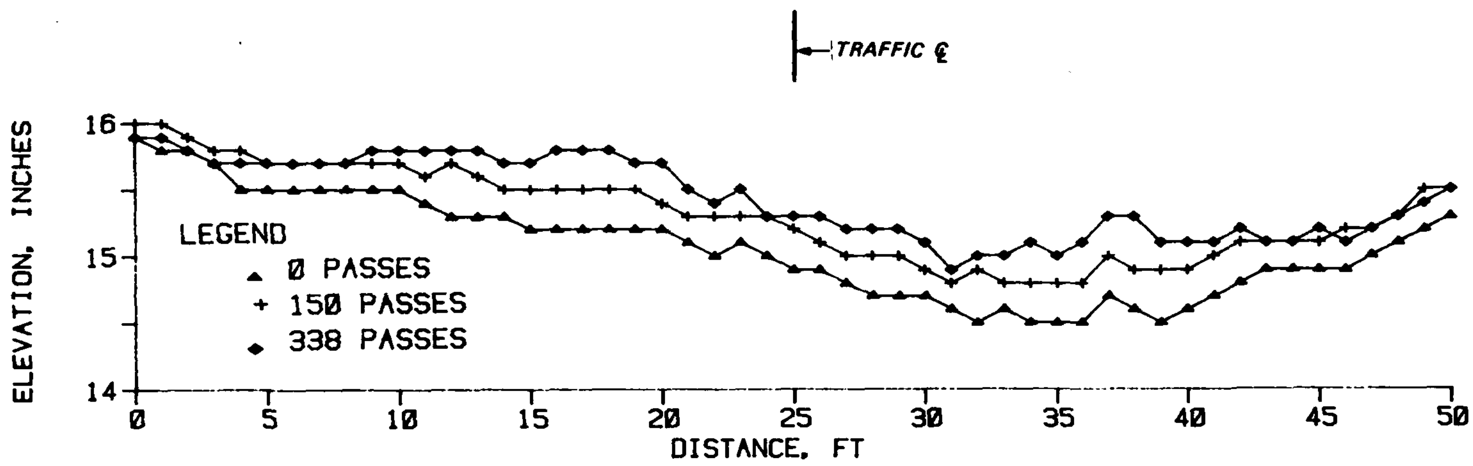


Figure B-12. Centerline Profile of Item 1

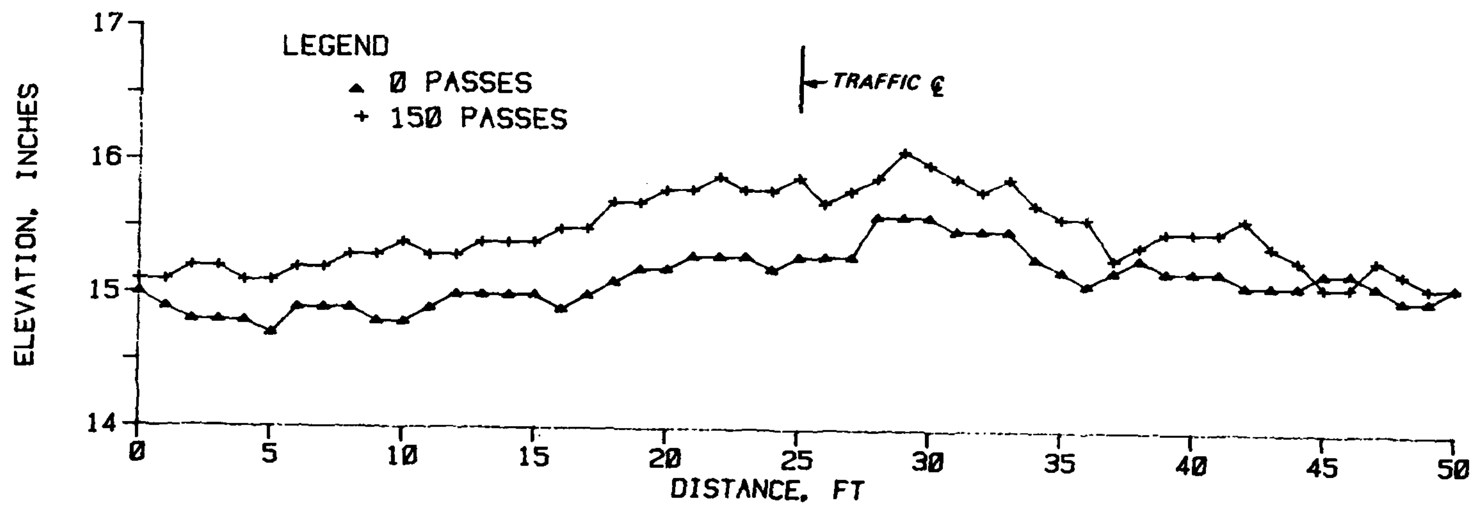


Figure B-13. Centerline Profile of Item 2

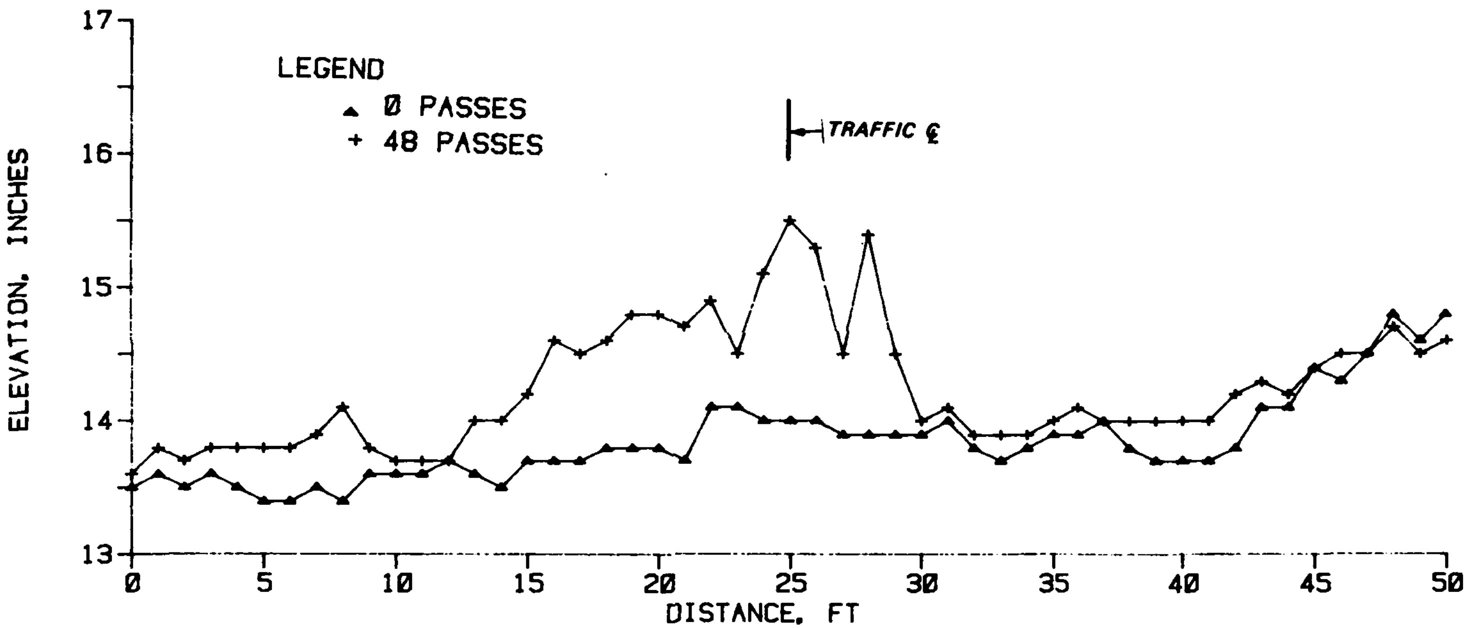


Figure B-14. Centerline Profile of Item 3

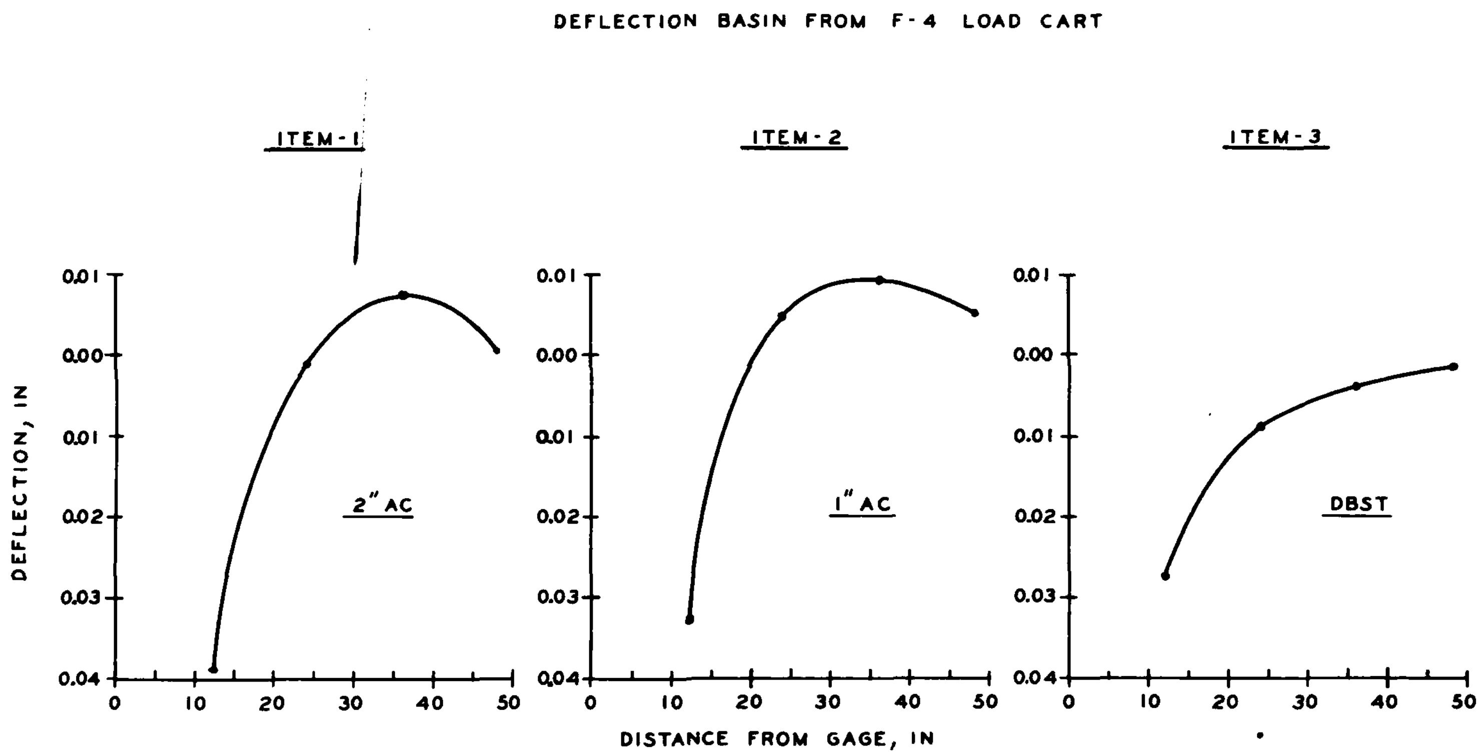


Figure B-15. Deflection Basins from F-4 Load Cart Measured with LVDTs

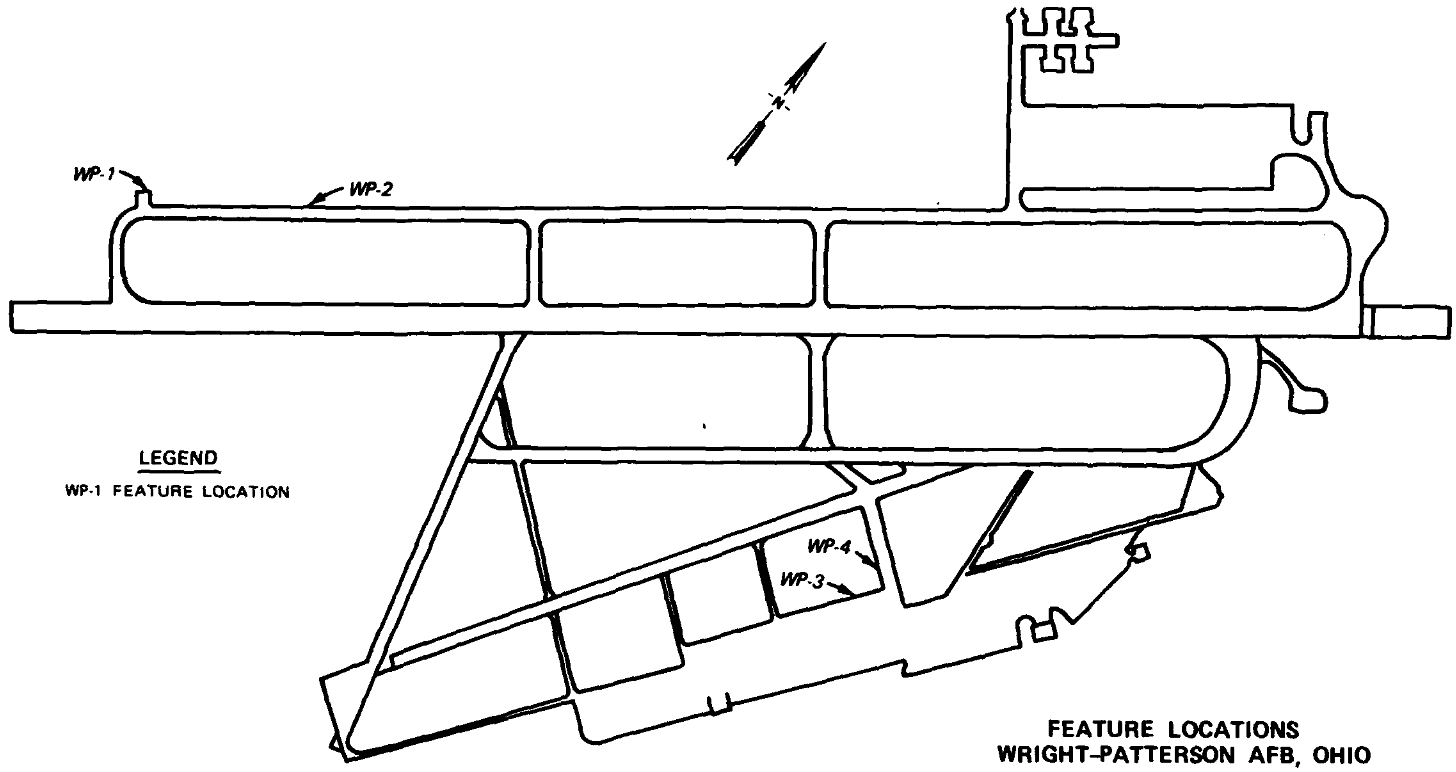


Figure B-16. Layout of Airfield Pavement Showing Locations of the Test Features

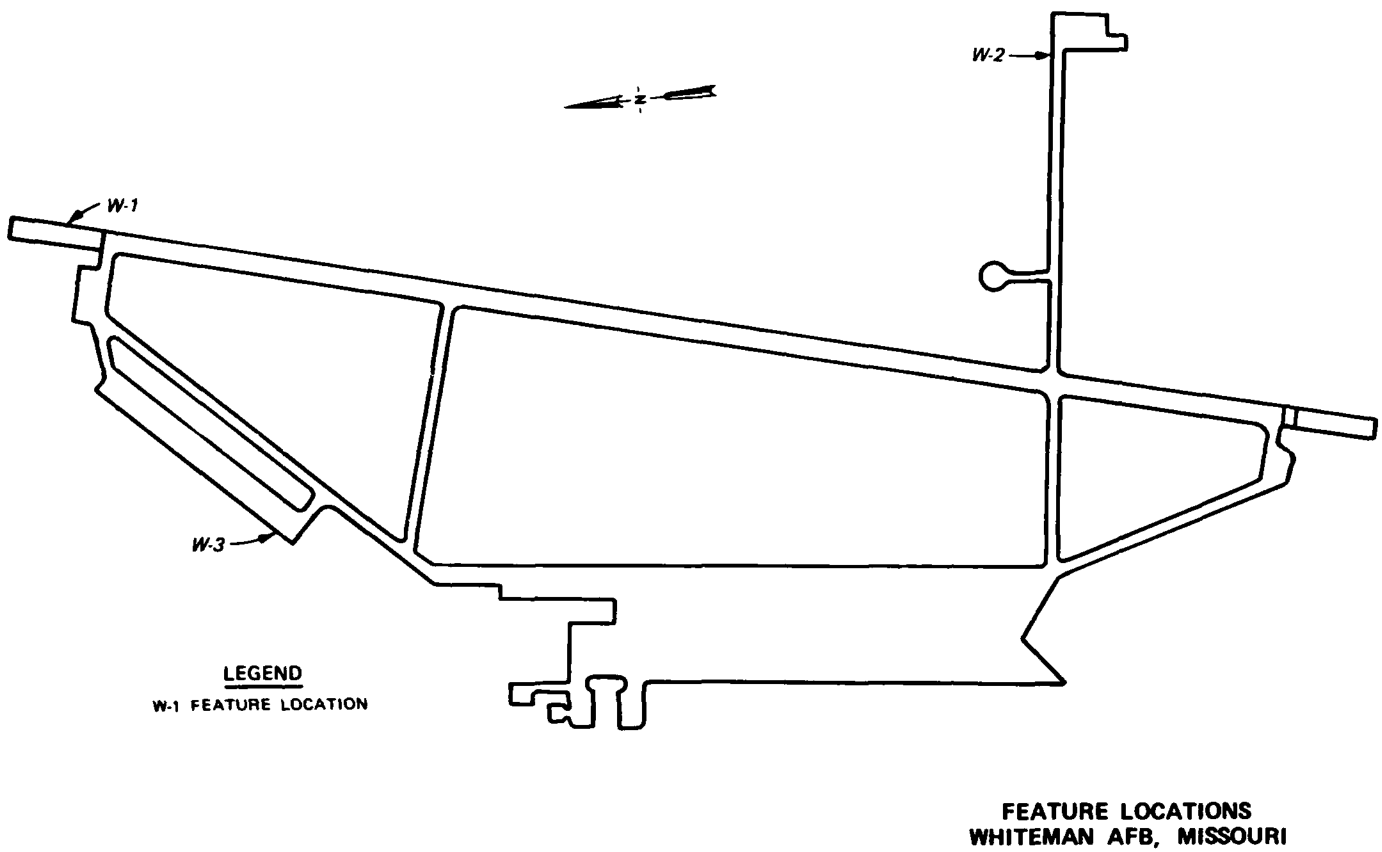
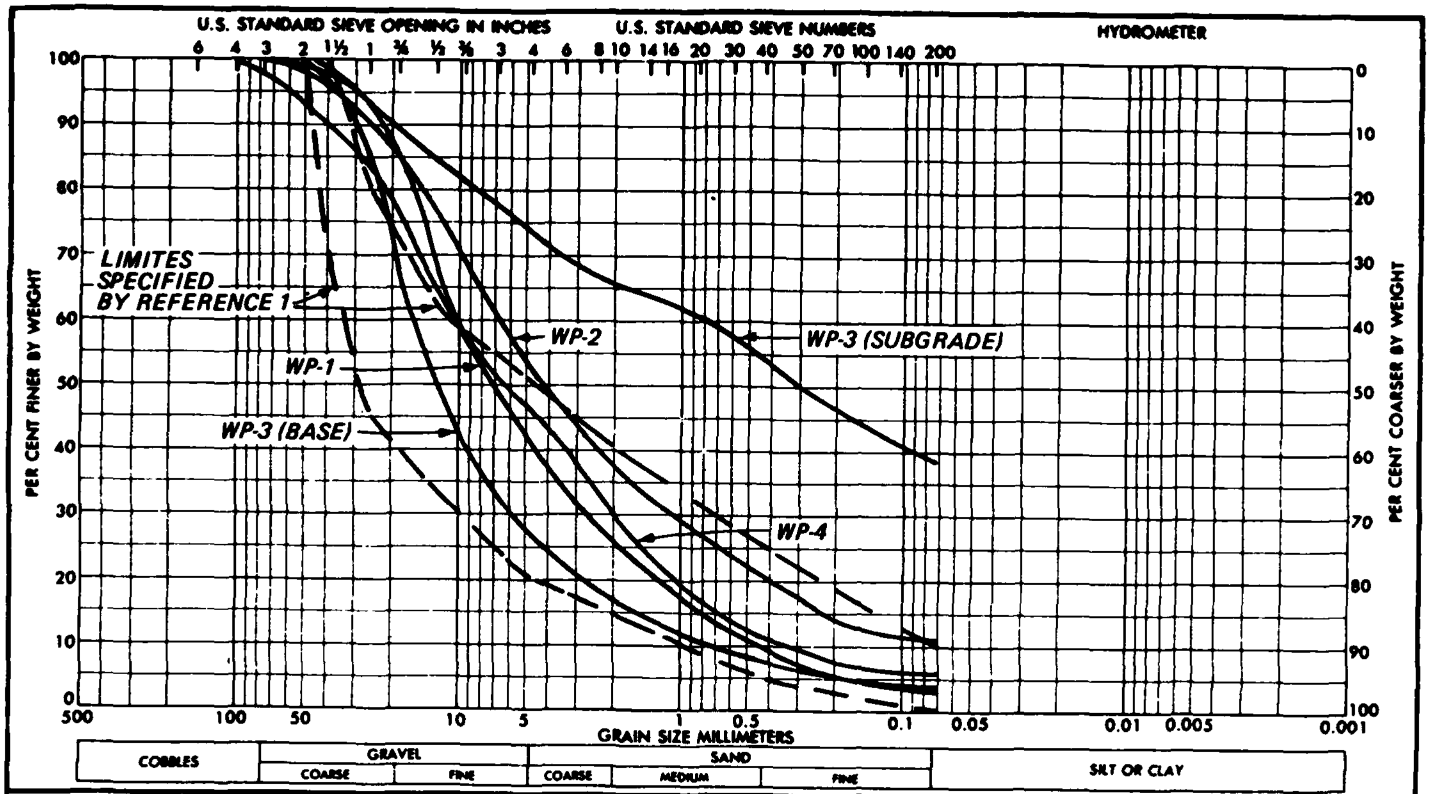


Figure B-17. Layout of Airfield Pavement Showing Locations of the Test Features



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
WP-1	BASE	GRAVEL (GW) BROWN		16	12	4
WP-2	BASE	CLAYEY GRAVEL (GP-GC) BROWN		20	14	6
WP-3	BASE	GRAVEL (GP) BROWN		17	13	4
WP-3	SUBGRADE	GRAVELLY CLAYEY SAND (SC)		31	15	16
WP-4	BASE	SILTY GRAVEL (GW-GM)		14	12	2

GRADATION CURVES

PROJECT ALTERNATE LAUNCH & RECOVERY SURFACES

AREA WRIGHT - PATTERSON AFB, OHIO

BORING NO

DATE

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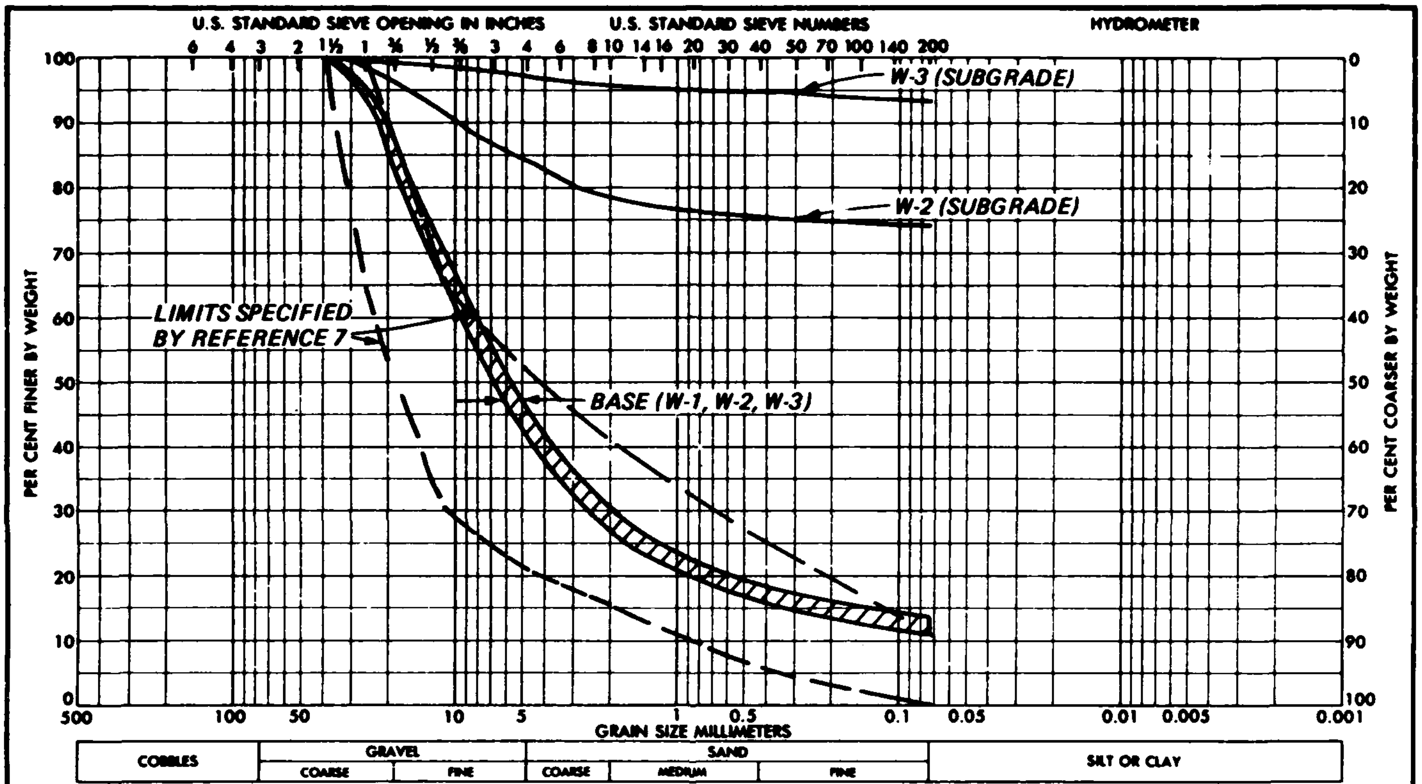
Figure B-18. Gradation Curves for Wright-Patterson AFB Material



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
W-1	BASE	SANDY CLAYEY GRAVEL (GC)		23	15	8
W-2	BASE	SANDY CLAYEY GRAVEL (GC)		23	14	9
W-3	BASE	SANDY CLAYEY GRAVEL (GC)		24	14	10
W-2	SUBGRADE	GRAVELLY SANDY CLAY (CH)		51	17	34
W-3	SUBGRADE	CLAY (CL)		49	17	32

GRADATION CURVES

PROJECT ALTERNATE LAUNCH & RECOVERY SURFACES

AREA WHITEMAN AFB, MO.

BORING NO.

DATE

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Figure B-19. Gradation Curves for Whiteman AFB Material

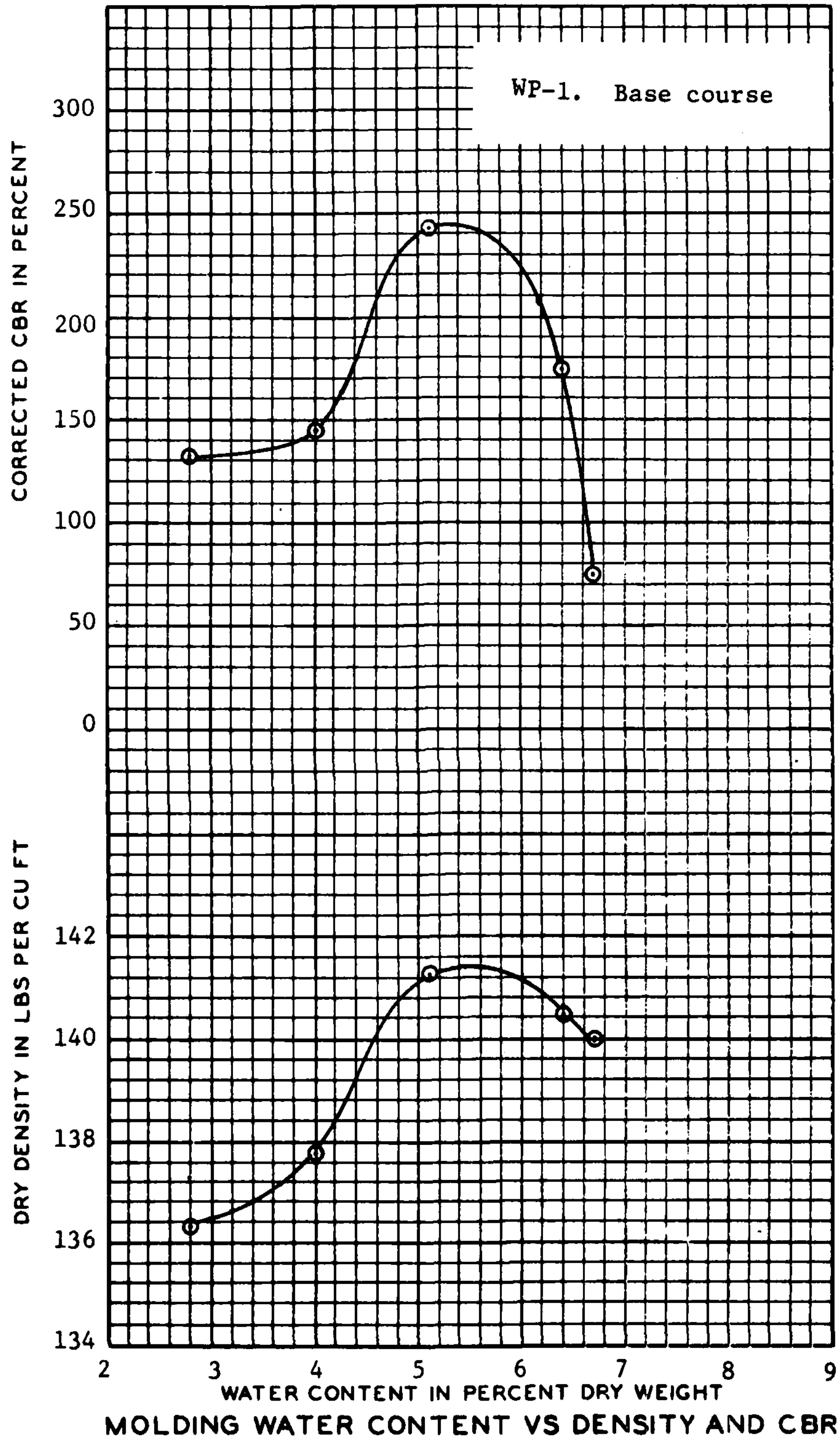


Figure B-20. Laboratory Compaction and Unsoaked CBRs for WP-1 Base Course

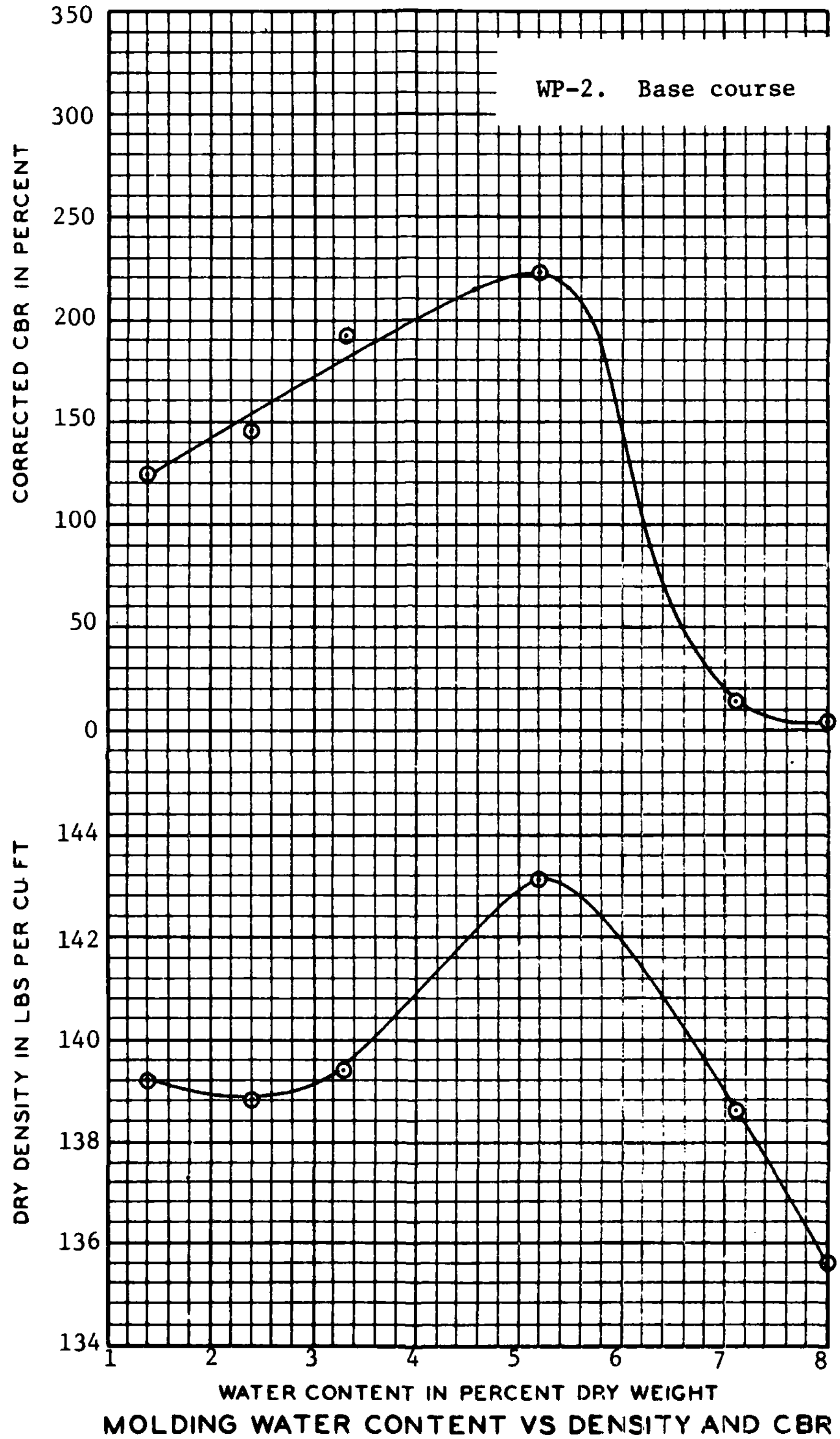


Figure B-21. Laboratory Compaction and Unsoaked CBRs for WP-2 Base Course

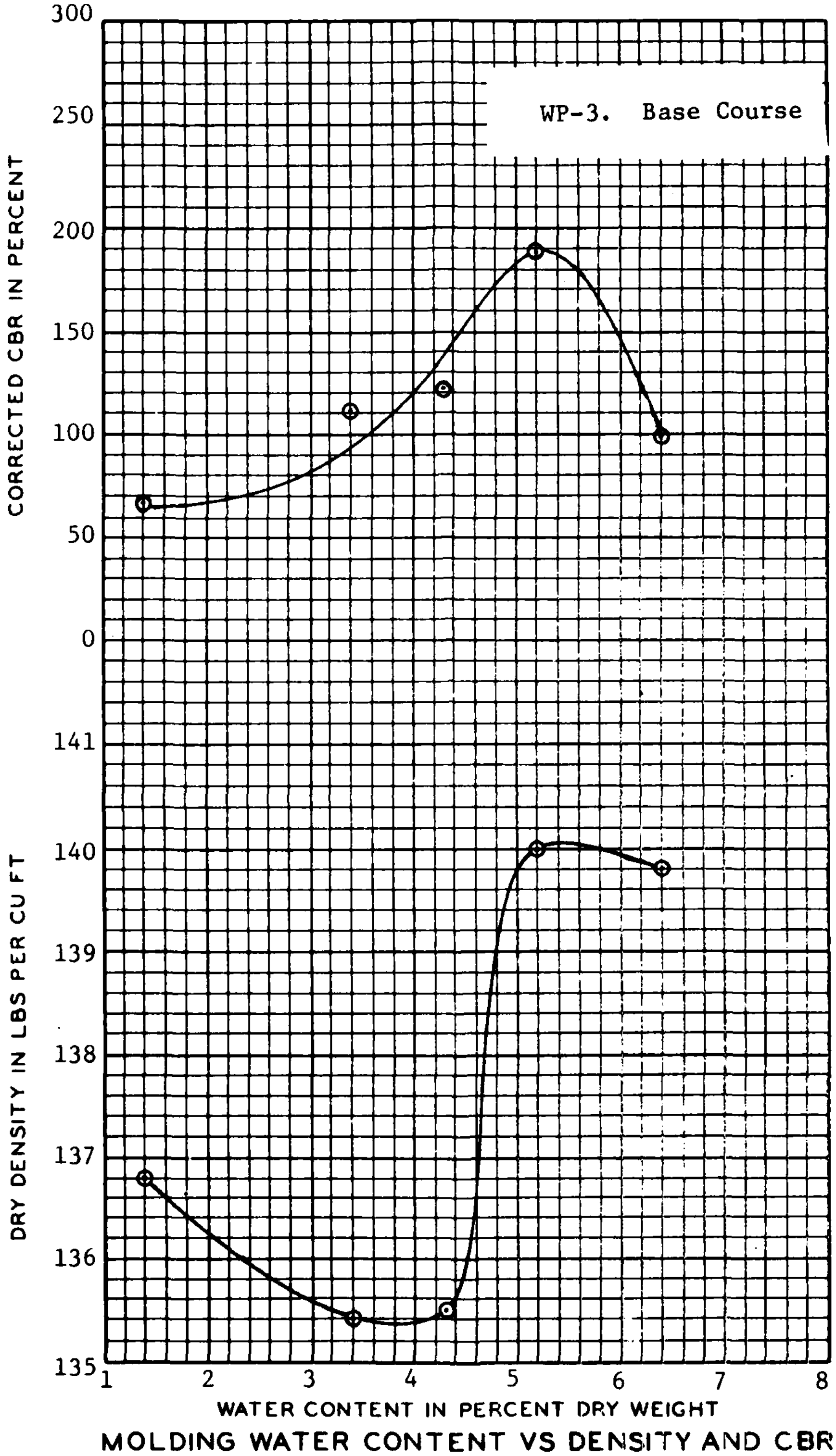


Figure B-22. Laboratory Compaction and Unsoaked CBRs for WP-3 Base Course

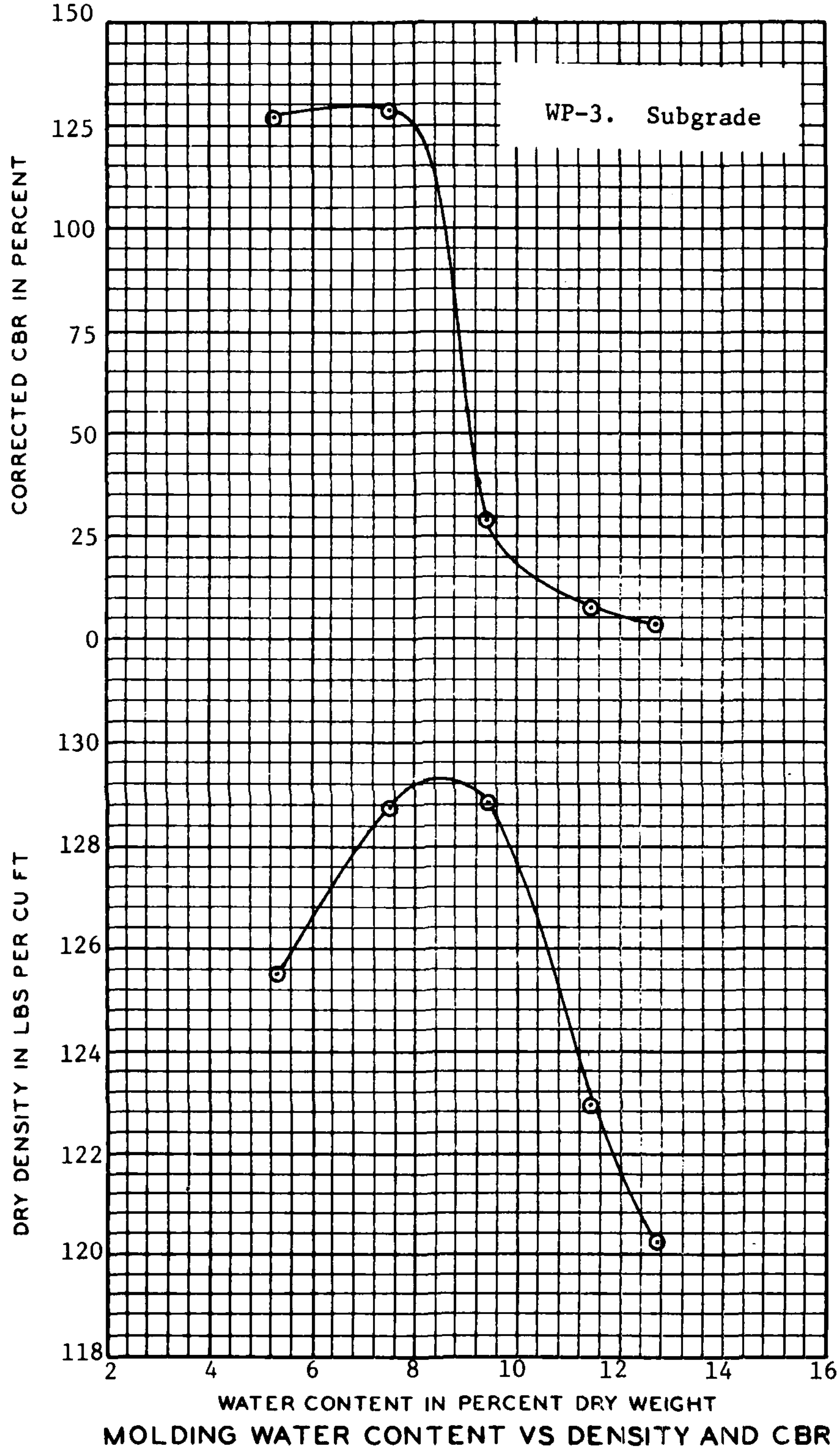


Figure B-23. Laboratory Compaction and Unsoaked CBRs for WP-3 Subgrade

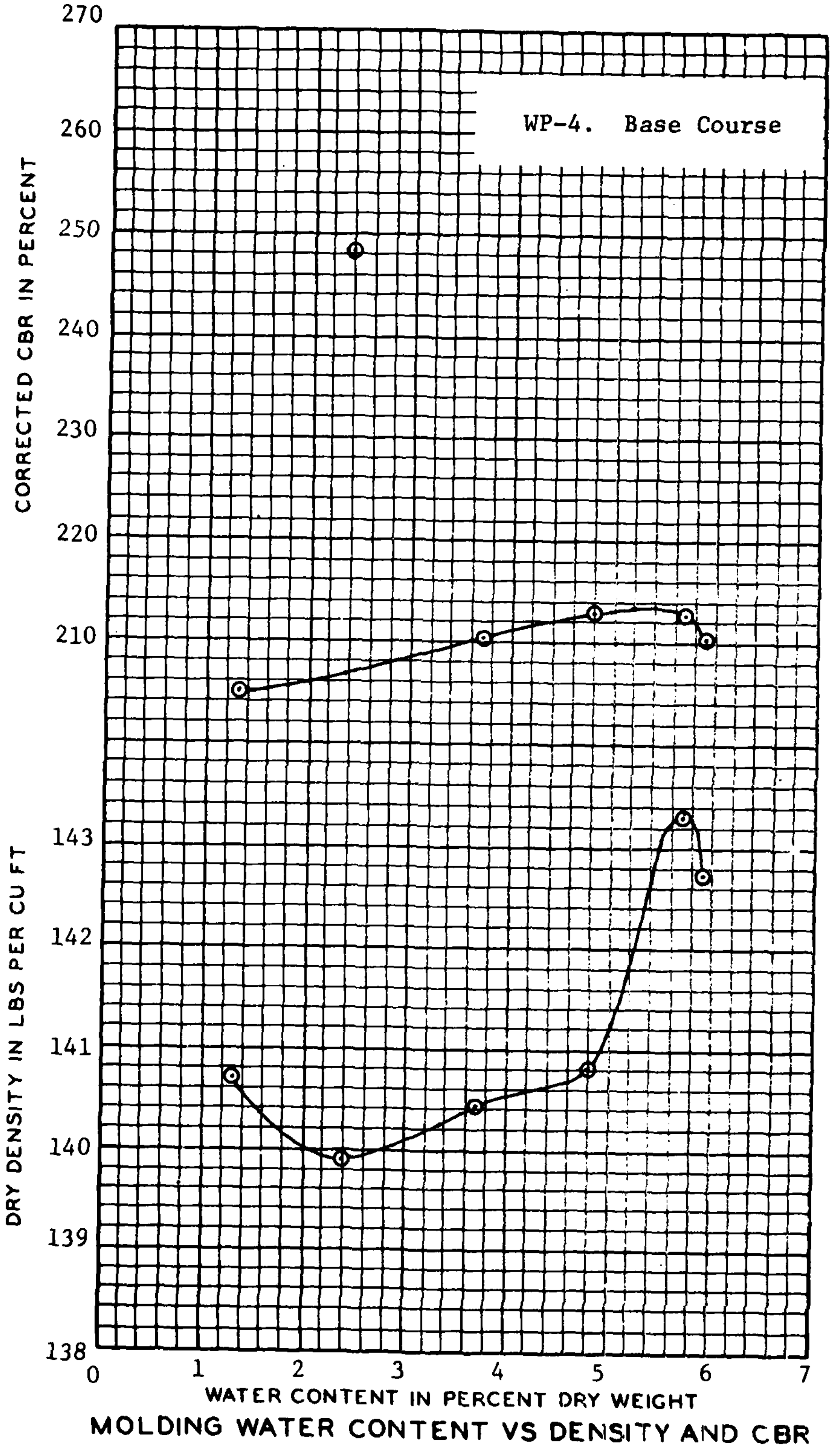


Figure B-24. Laboratory Compaction and Unsoaked CBRs for WP-4 Base Course

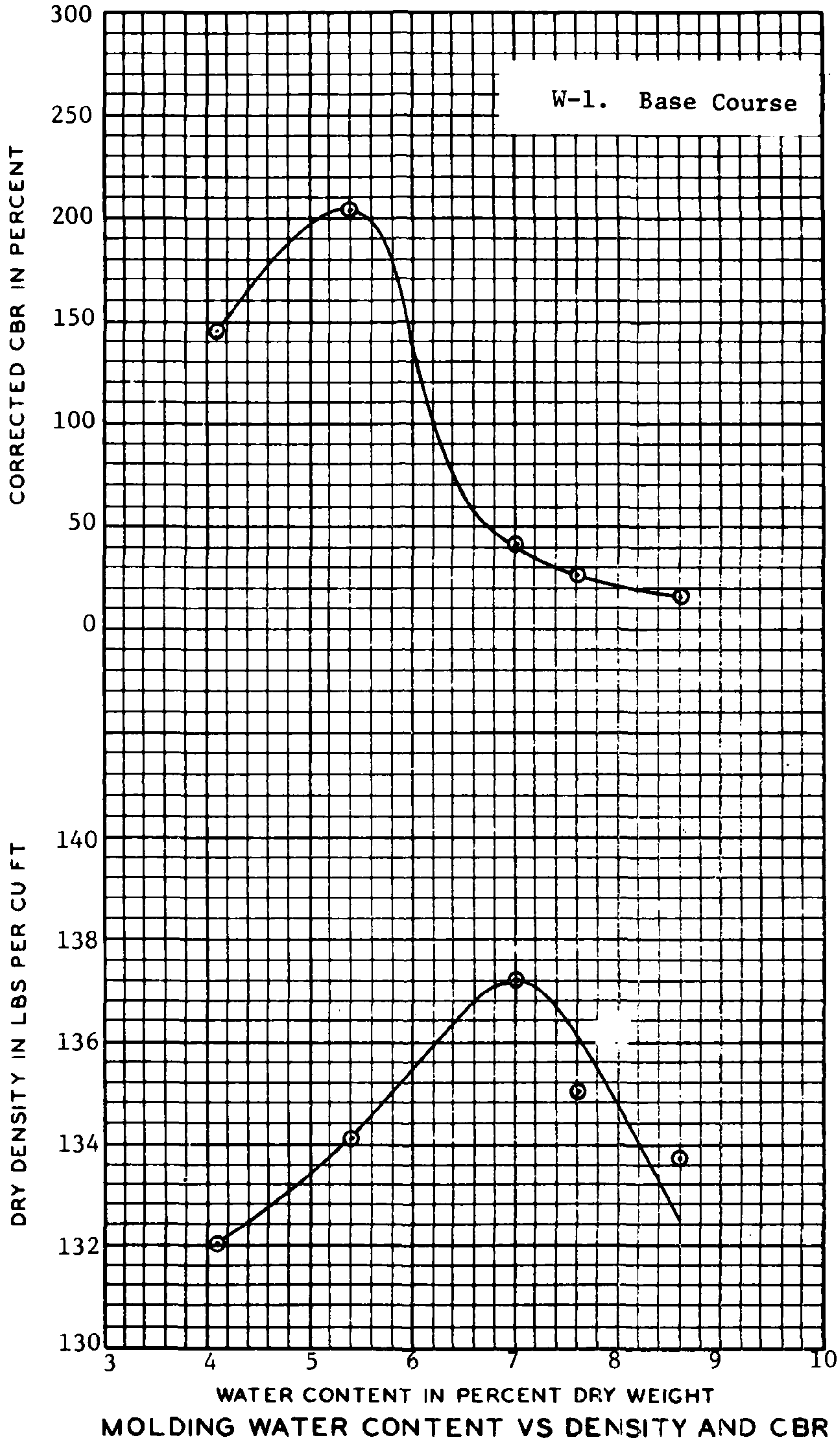


Figure B-25. Laboratory Compaction and Unsoaked CBRs for W-1 Base Course

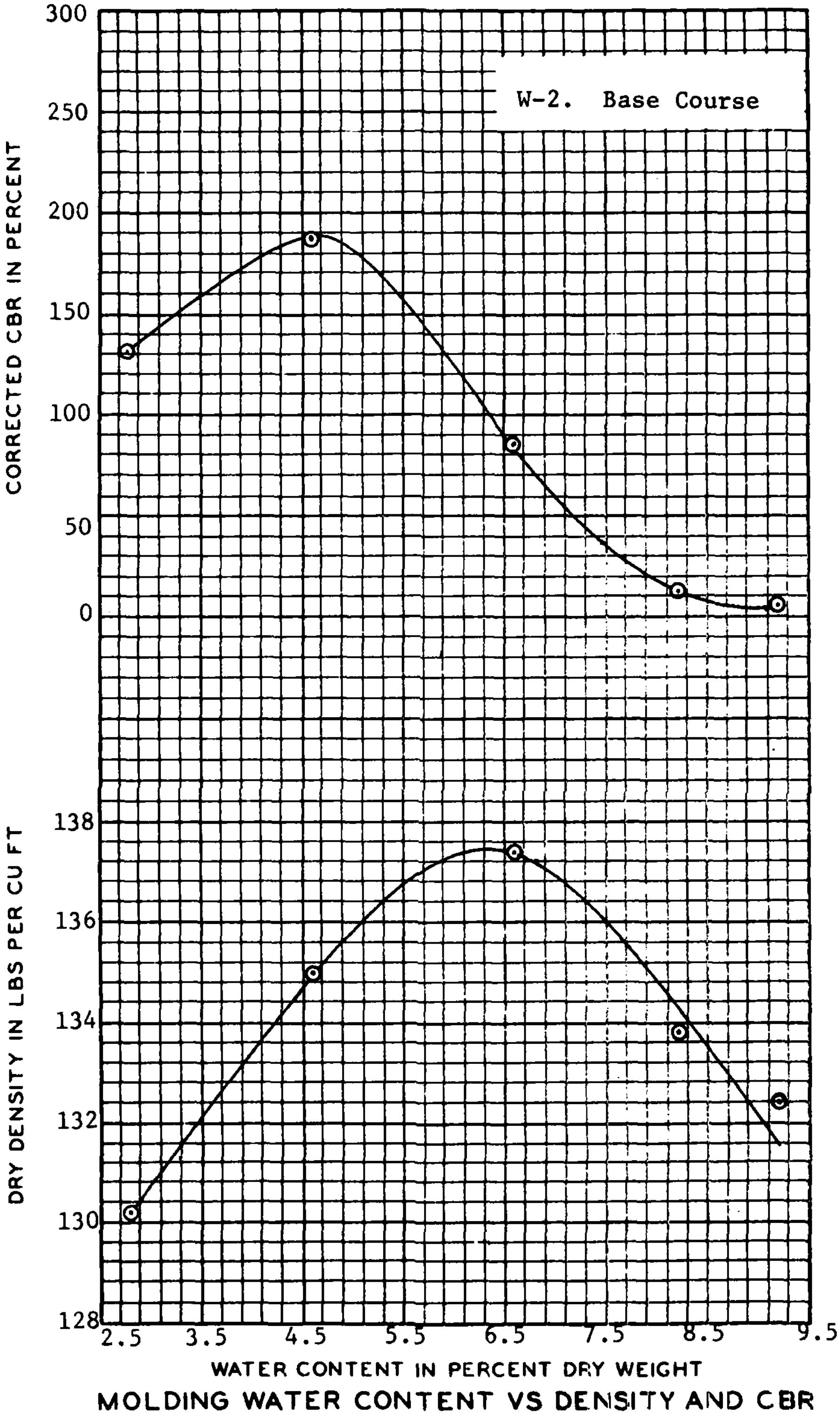


Figure B-26. Laboratory Compaction and Unsoaked CBRs for W-2 Base Course

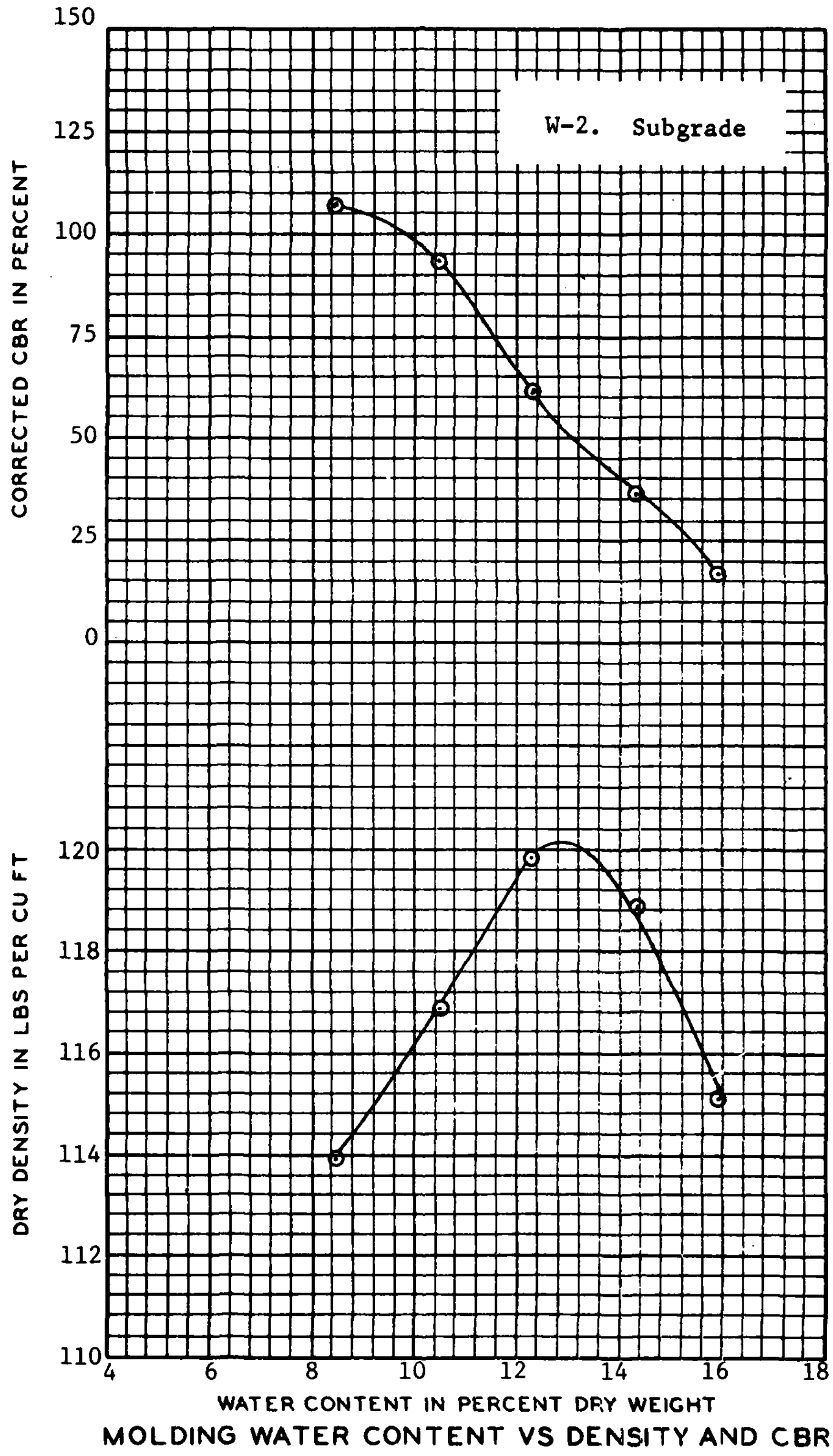


Figure B-27. Laboratory Compaction and Unsoaked CBRs for W-2 Subgrade

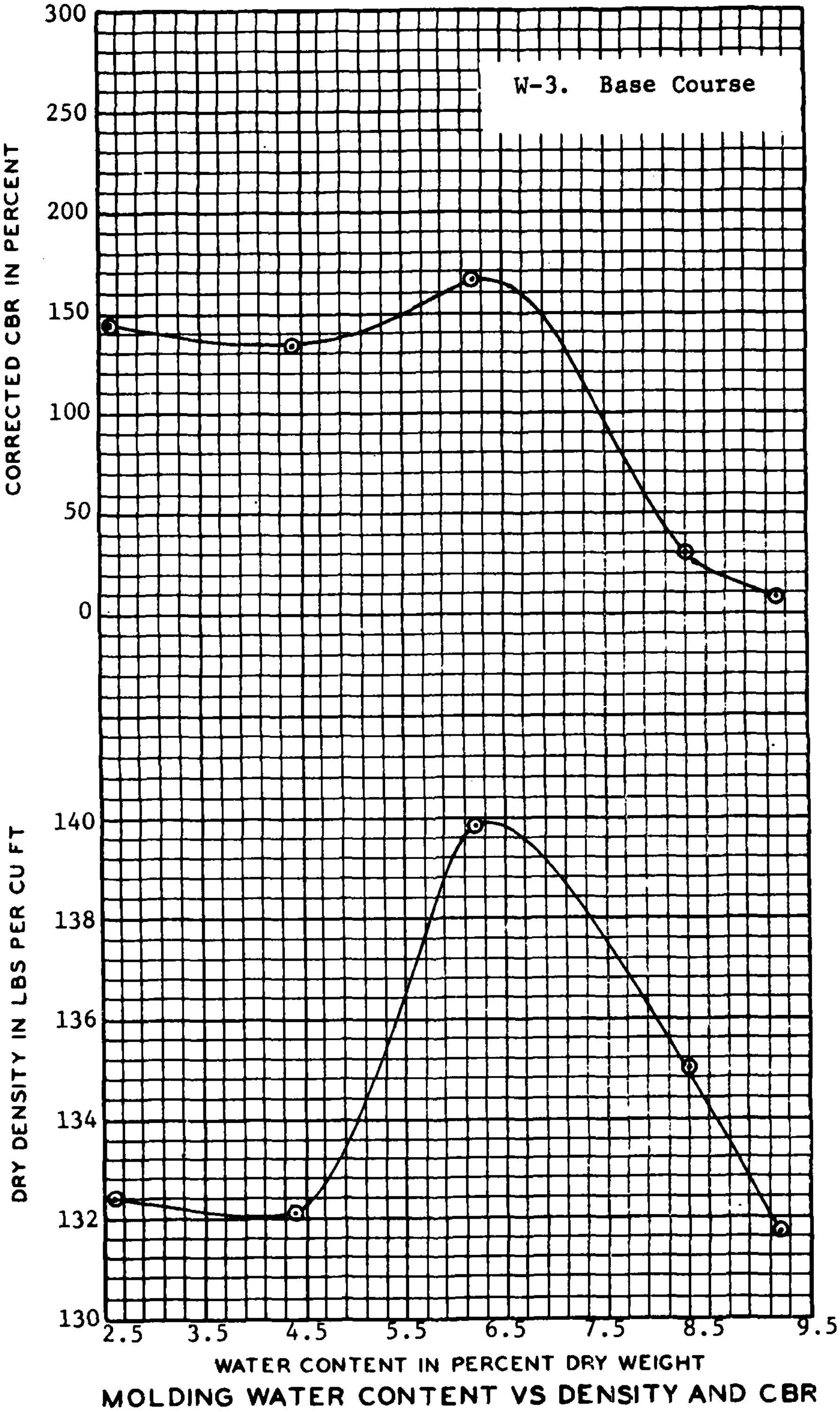


Figure B-28. Laboratory Compaction and Unsoaked CBRs for W-3 Base Course

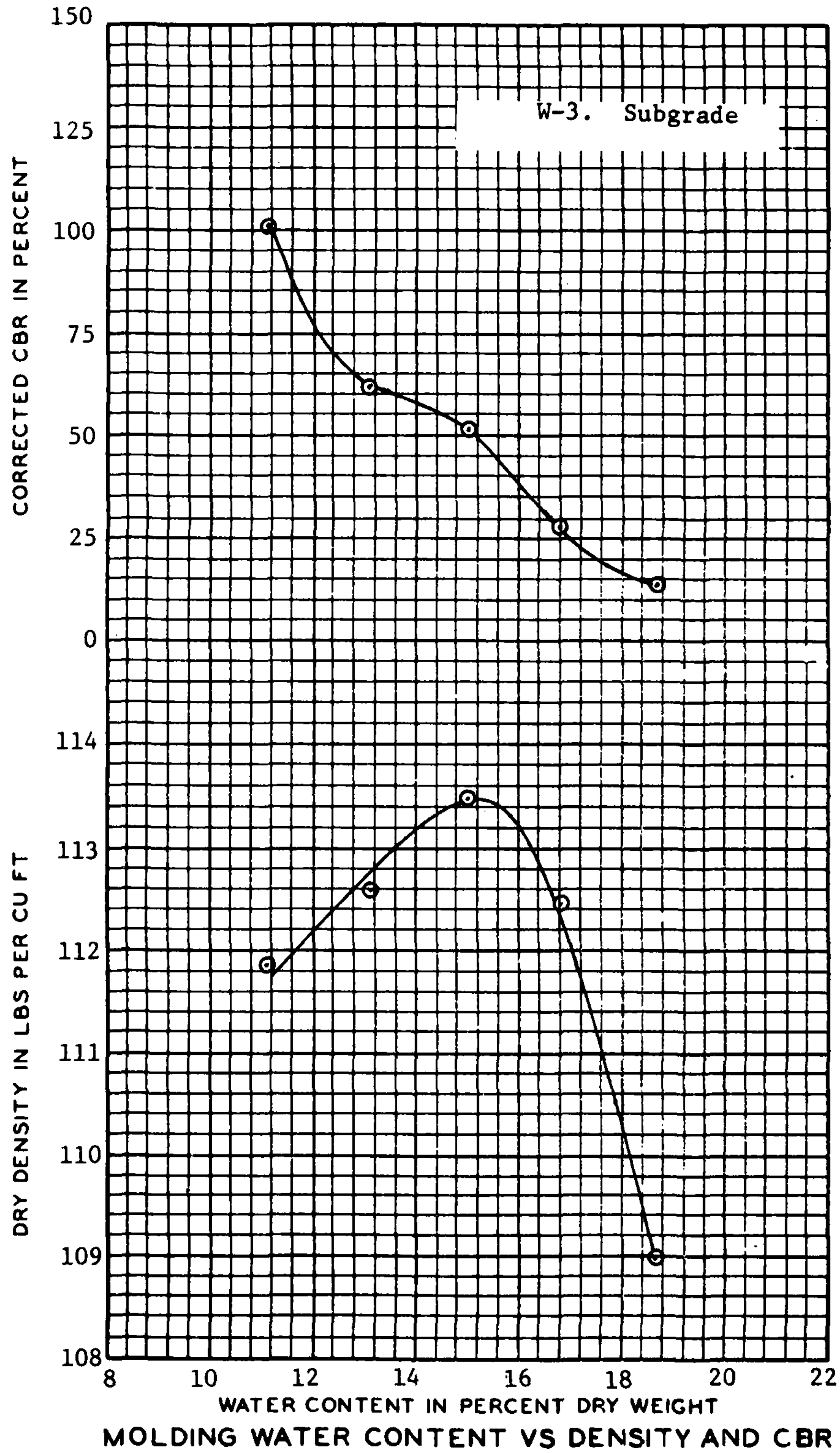
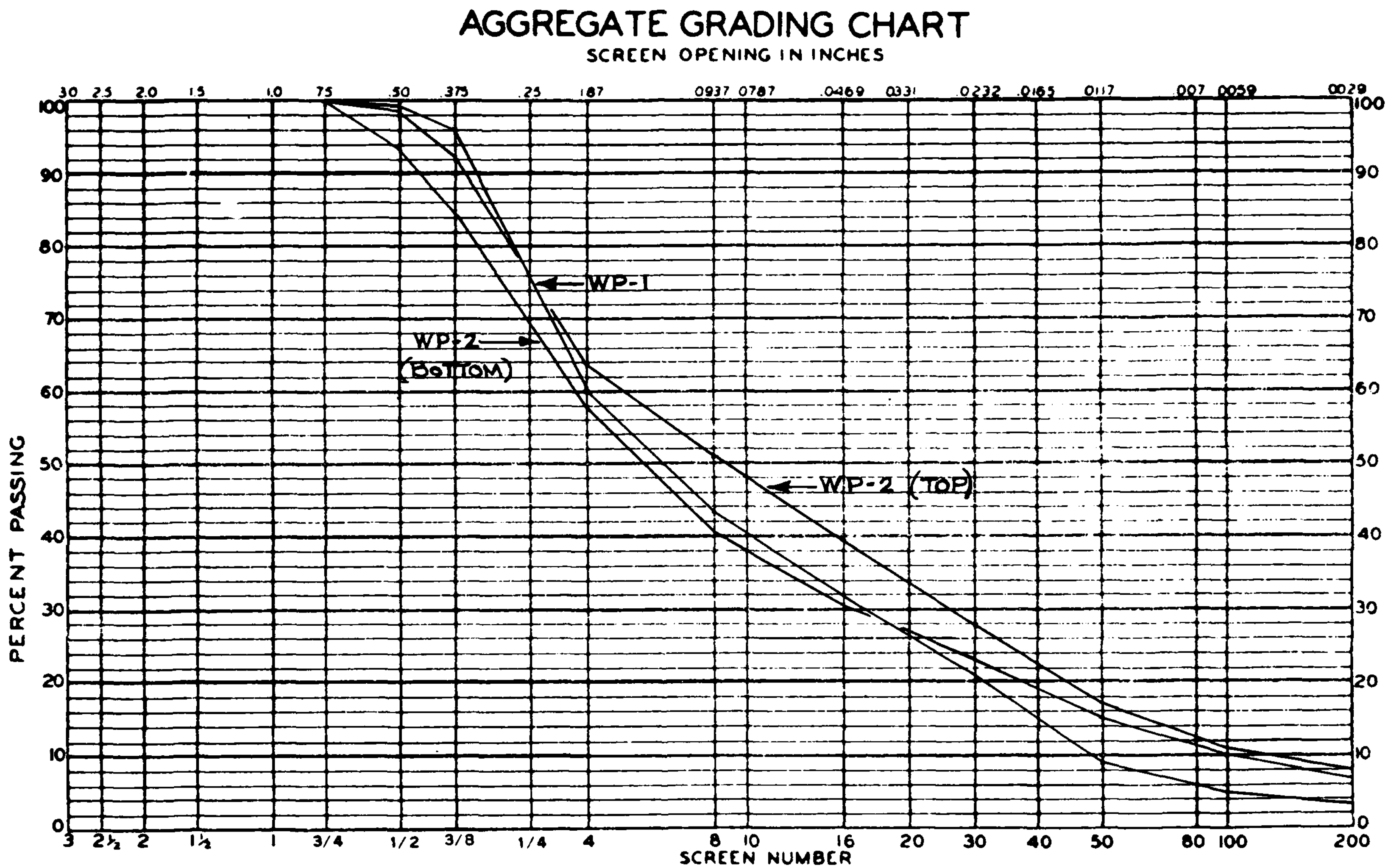


Figure B-29. Laboratory Compaction and Unsoaked CBRs for W-3 Subgrade



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SAMPLE IDENTIFICATION

Wright-Patterson Air Force Base, Ohio

WES FORM 886
AUGUST 55

Figure B-30. Gradations for Wright-Patterson AFB Asphalt Mixtures

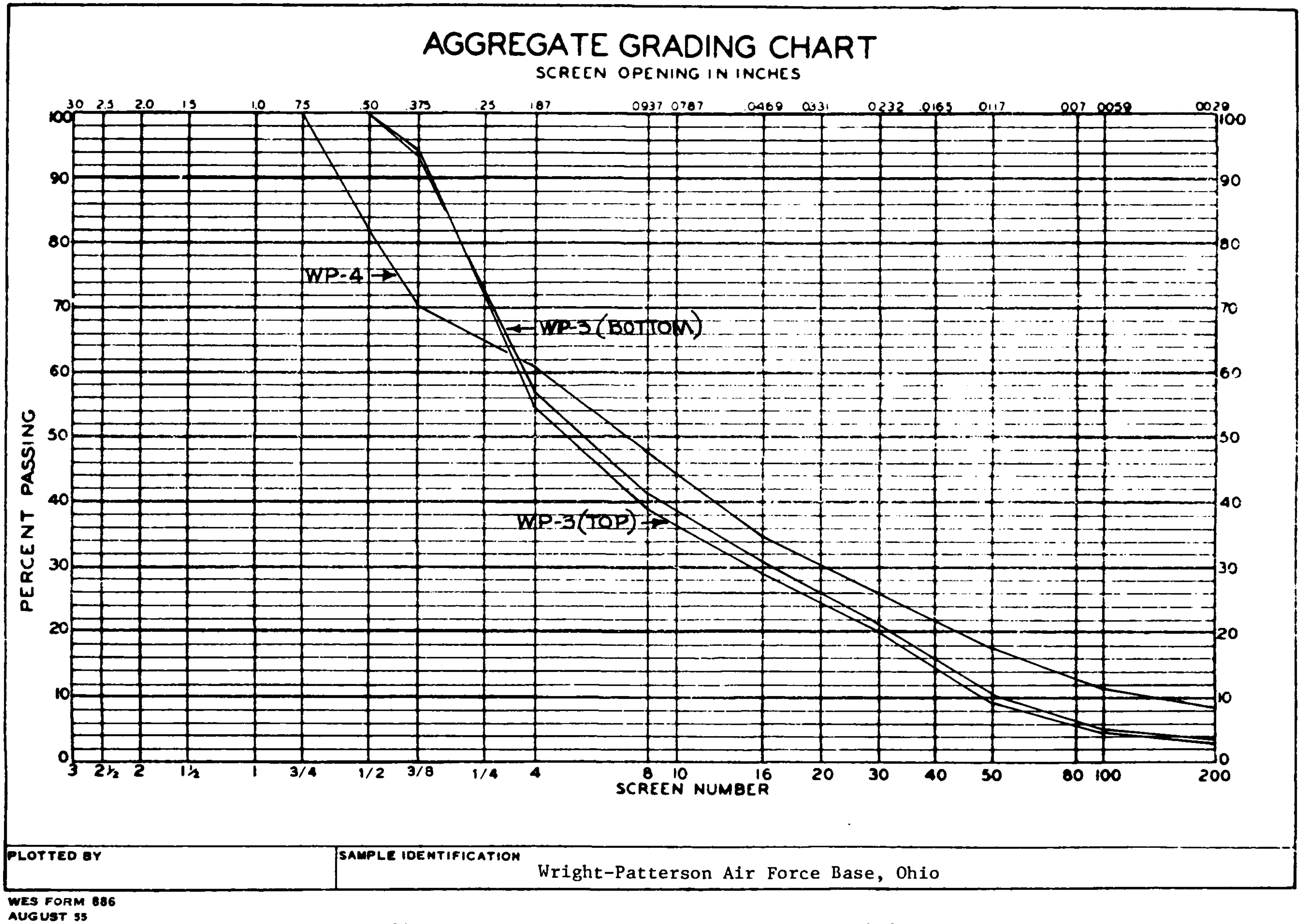


Figure B-31. Gradations for Wright-Patterson AFB Asphalt Mixtures

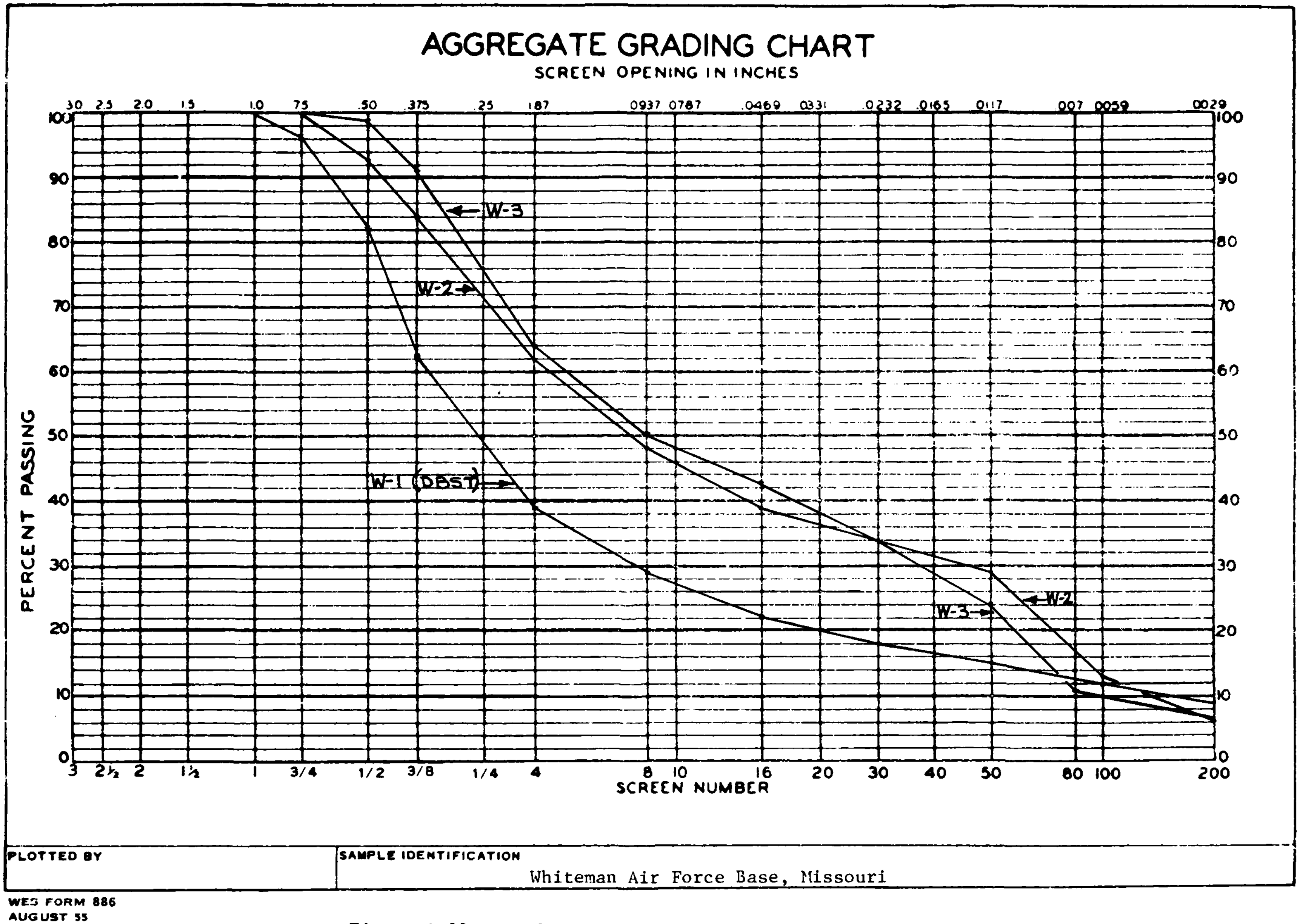
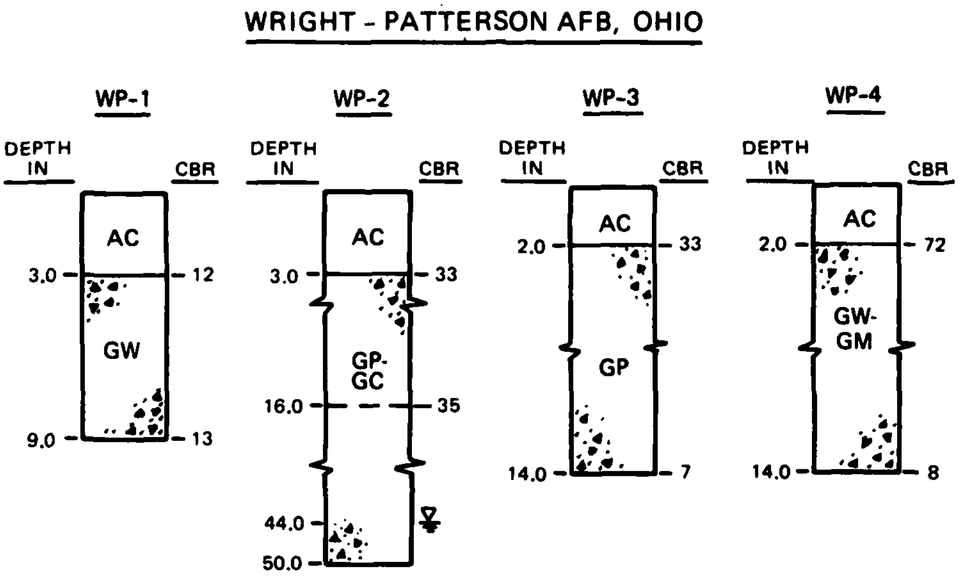


Figure B-32. Gradations for Whiteman AFB Asphalt Mixtures

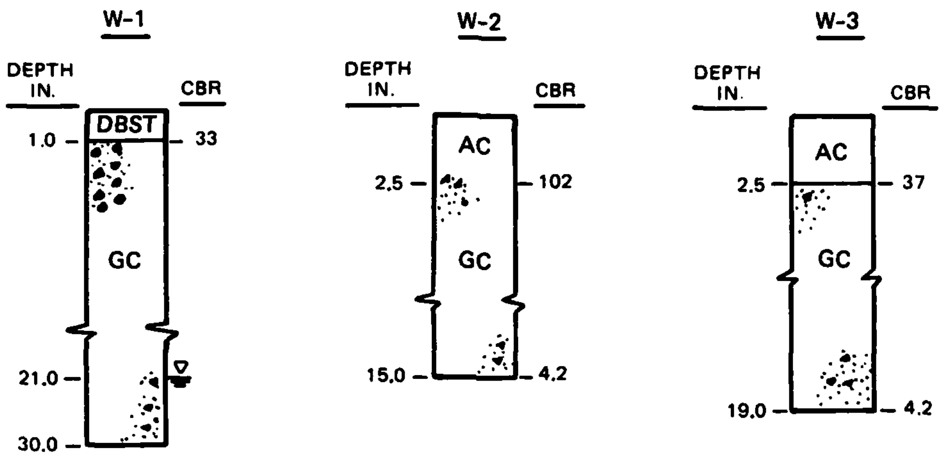


	WP-1	WP-2	WP-3	WP-4
PASSES TO 1-IN RUT	39	400	60	100
PASSES TO 3-IN RUT	44	643	90	162
SKIDS TO FAILURE	1	6*	6	35
TURNS TO FAILURE	11	75*	28	75*

* NO FAILURE

Figure B-33. Structure and Traffic Test Results of Wright-Patterson AFB Pavements

WHITEMAN AFB, MO.



	W-1	W-2	W-3
PASSES TO 1-IN RUT	105	25	33
PASSES TO 3-IN RUT	28[illegible]	132	86
SKIDS TO FAILURE	1	3	
TURNS TO FAILURE	75*	27	27

* NO FAILURE

Figure B-34. Structure and Traffic Test Results of Whiteman AFB Pavements

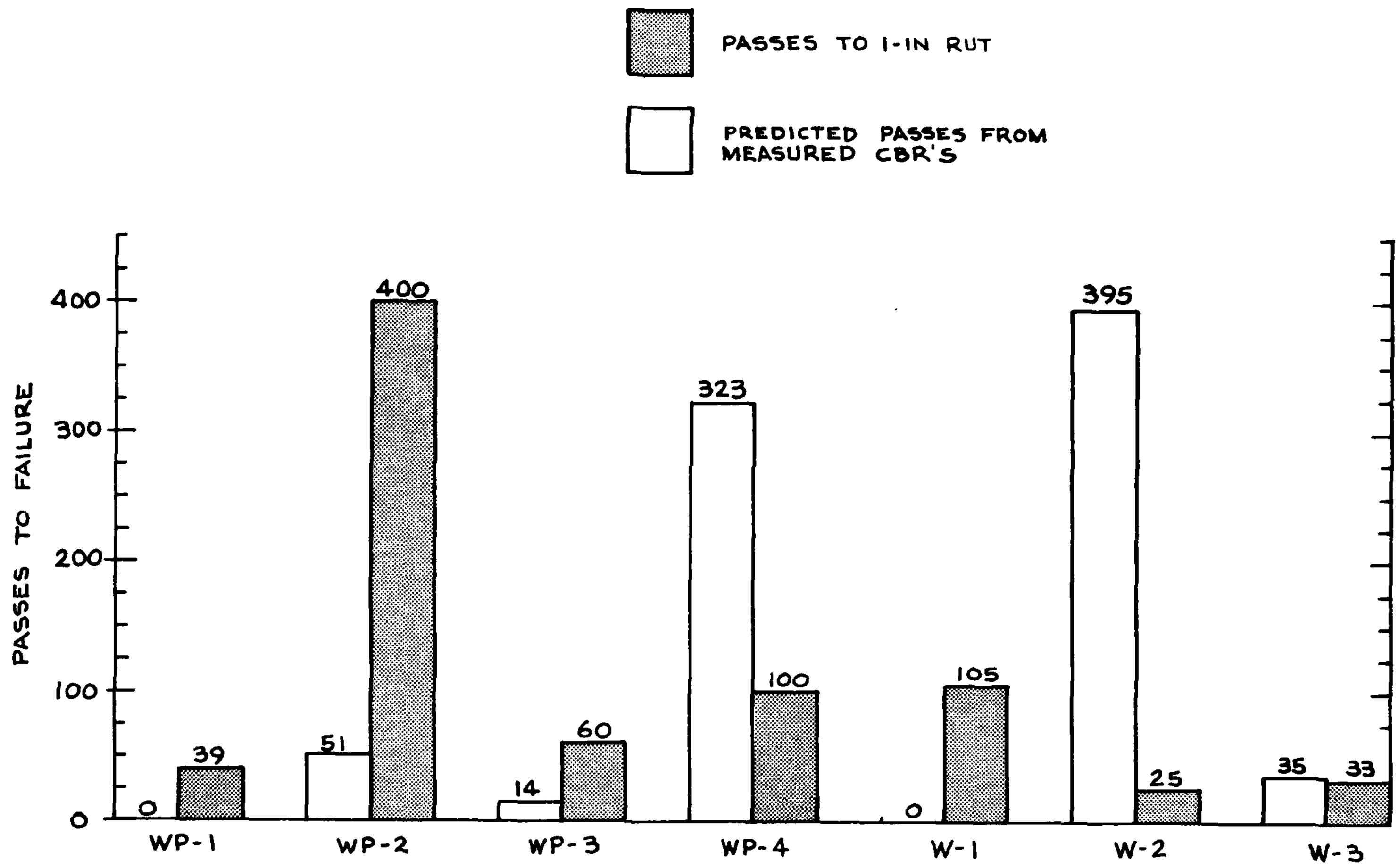


Figure B-35. Comparison of Predicted Passes to Failure from In-Place Measured CBRs to Traffic Results

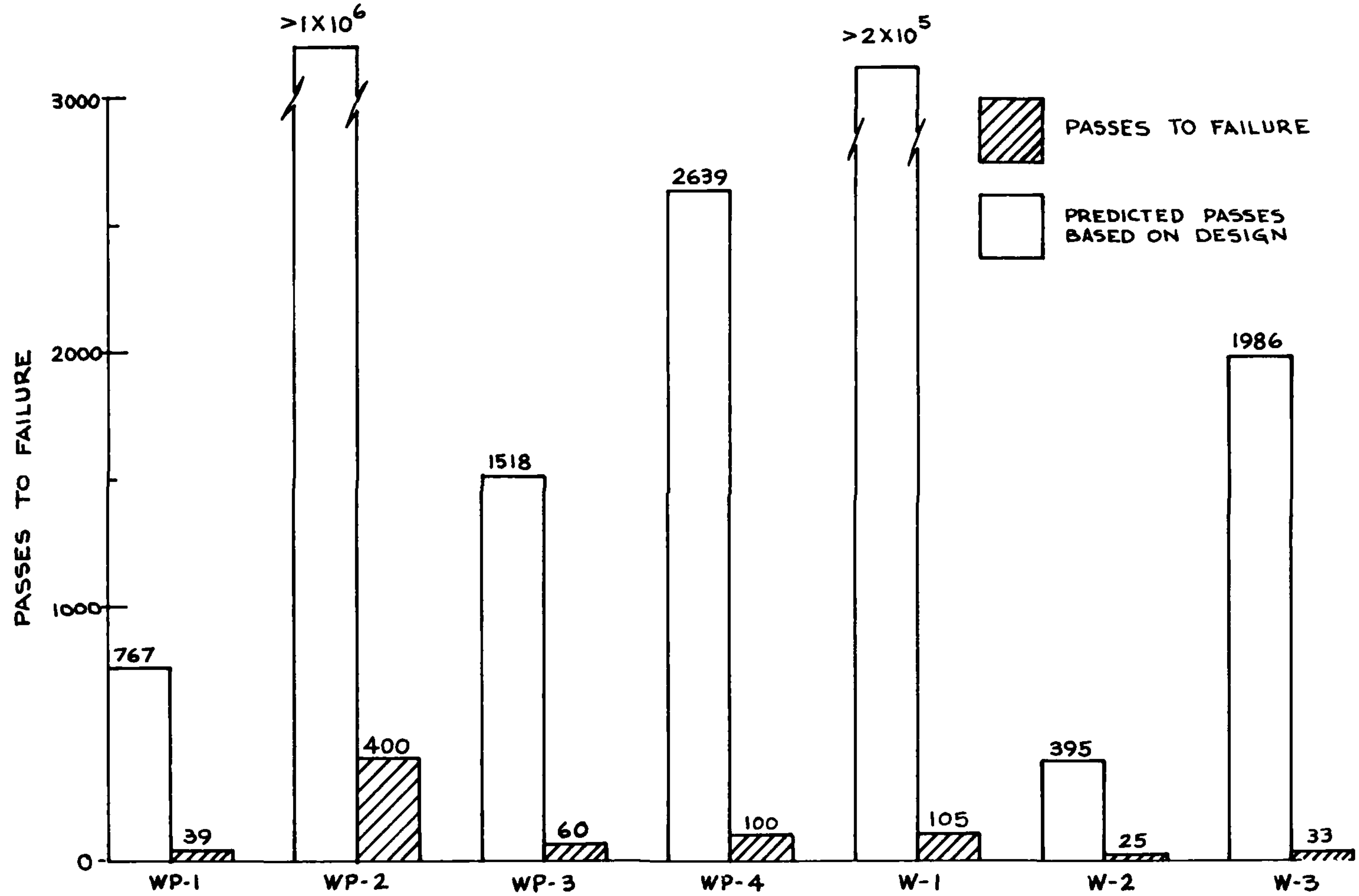


Figure B-36. Comparison of Design Passes to Failure to Traffic Results

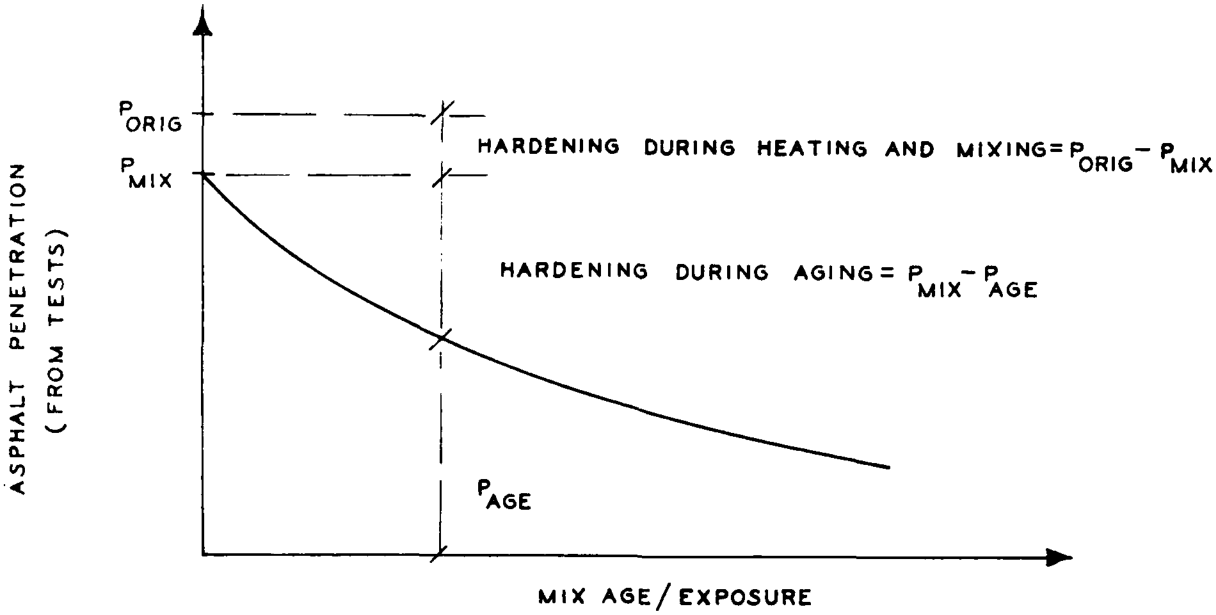


Figure B-37. Asphalt Hardening as a Result of Age

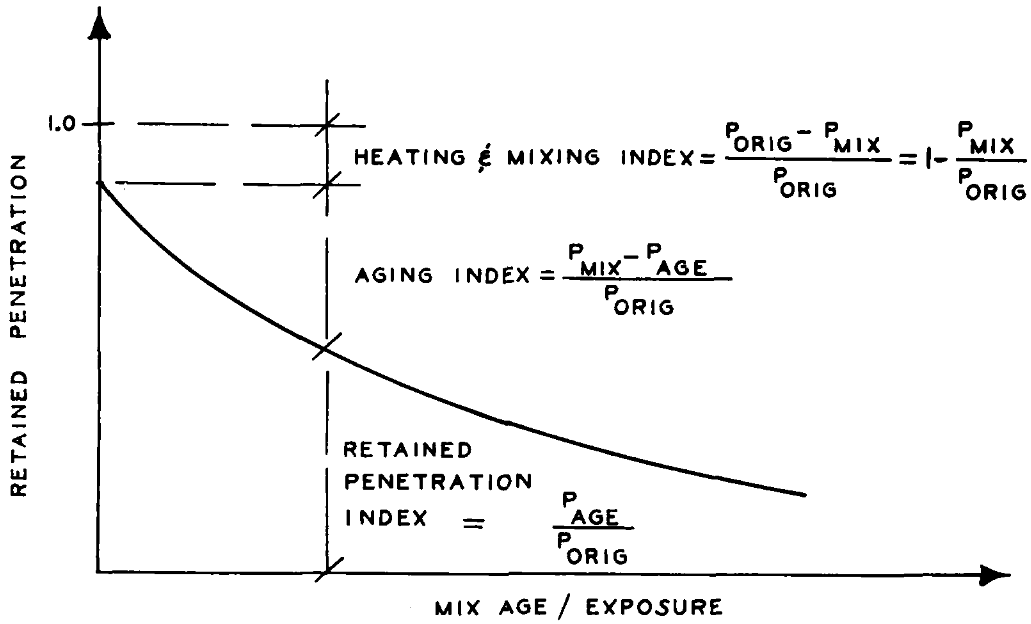
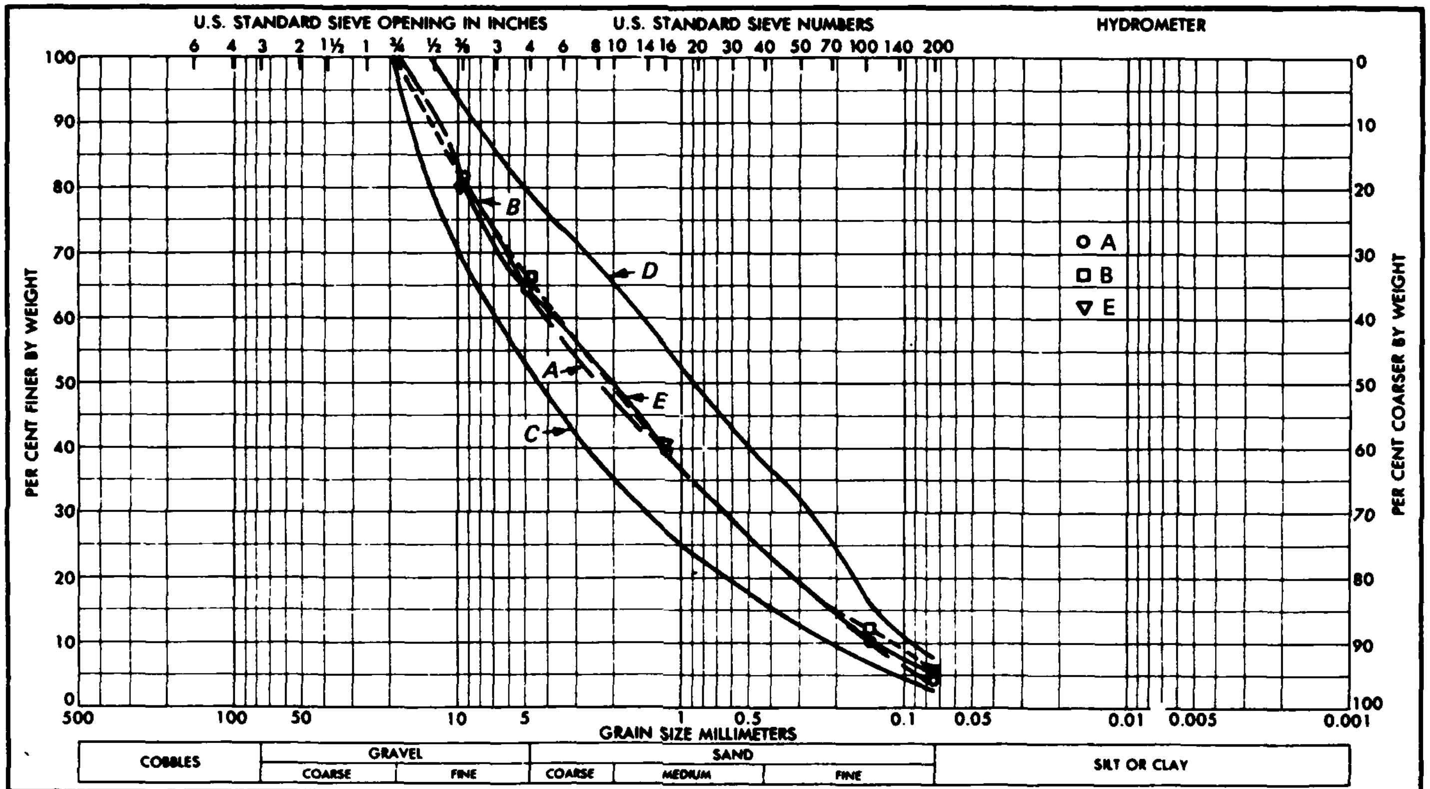


Figure B-38. Asphalt-Hardening Indices Based on Penetration



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI	PROJEC ALRS AGING TESTS
A, B, E		MEDIUM					
C		COARSE					AREA
D		FINE					BORING NO
GRADATION CURVES							DATE

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1 MAY 63

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Figure B-39. Aggregate Gradations

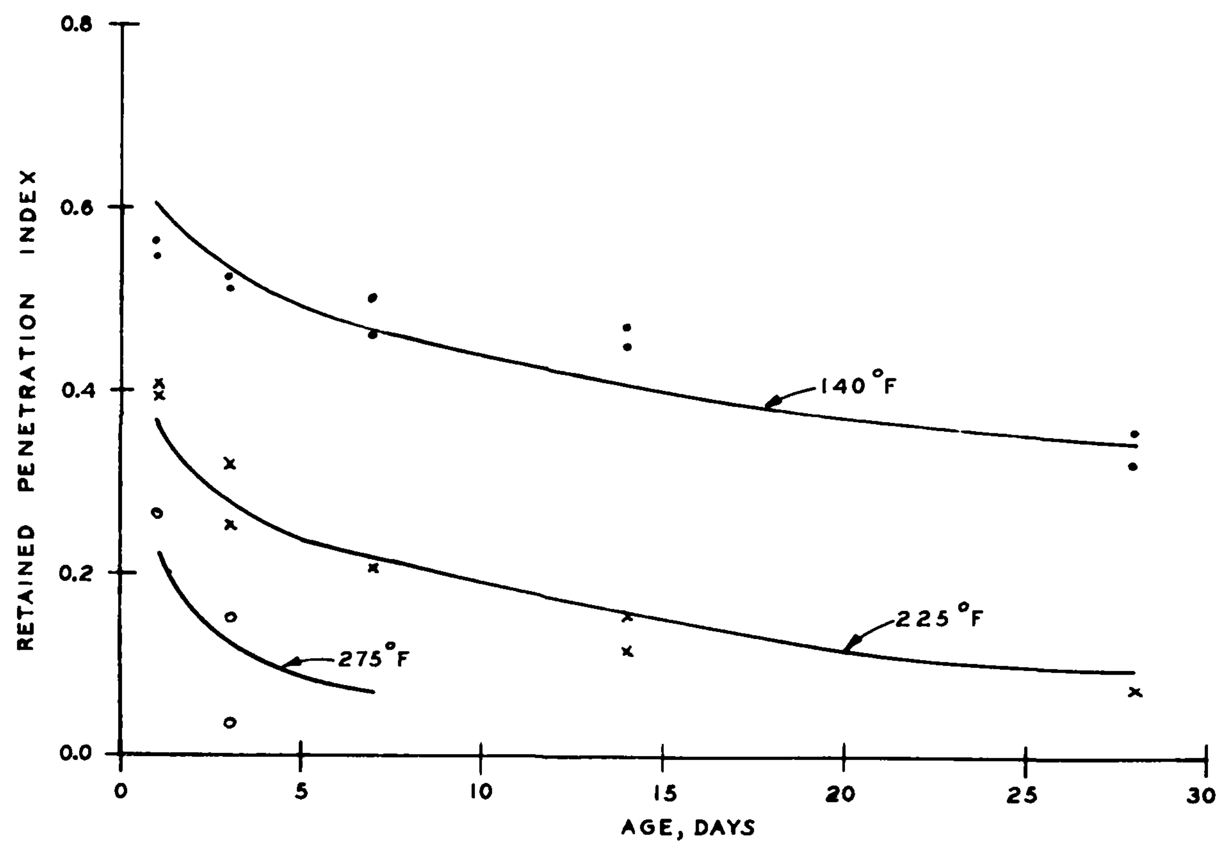


Figure B-40. General Aging Test Results for Three Temperatures

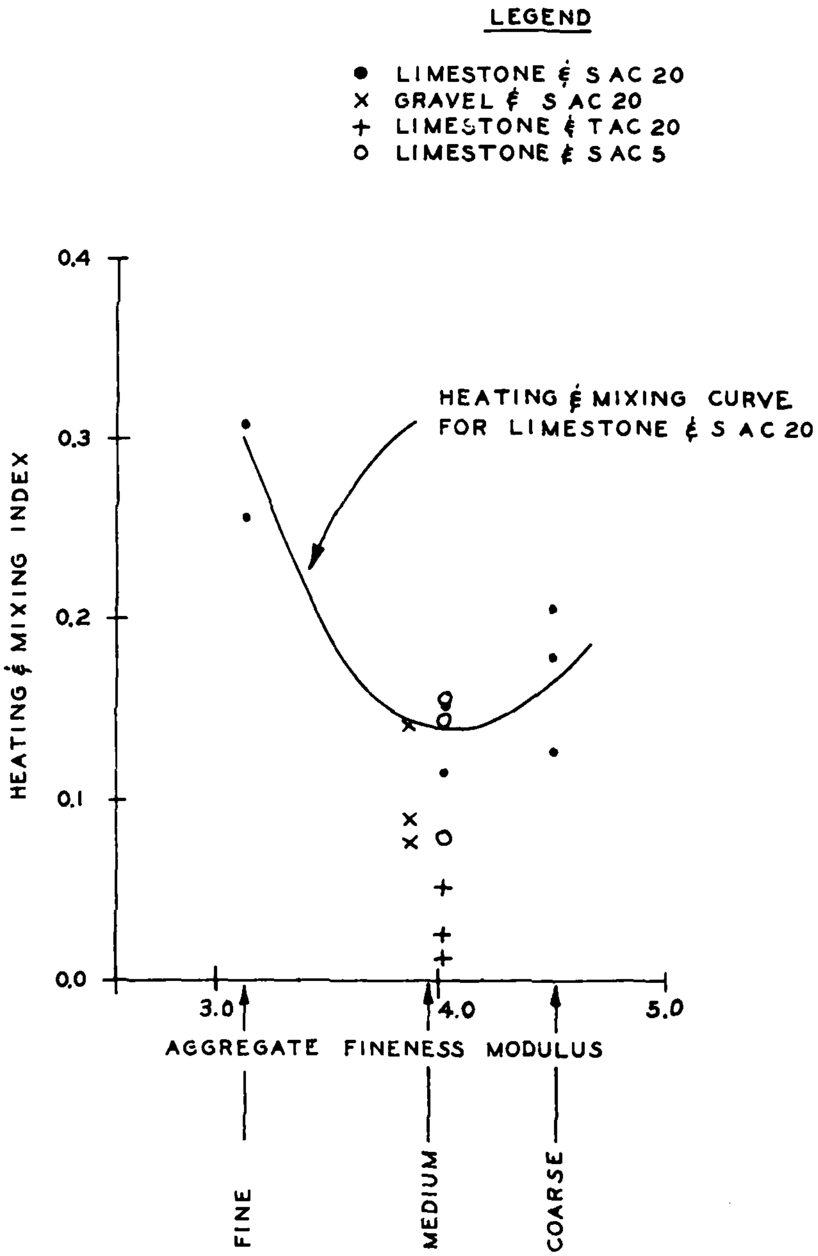


Figure B-41. Heating and Mixing Relationship

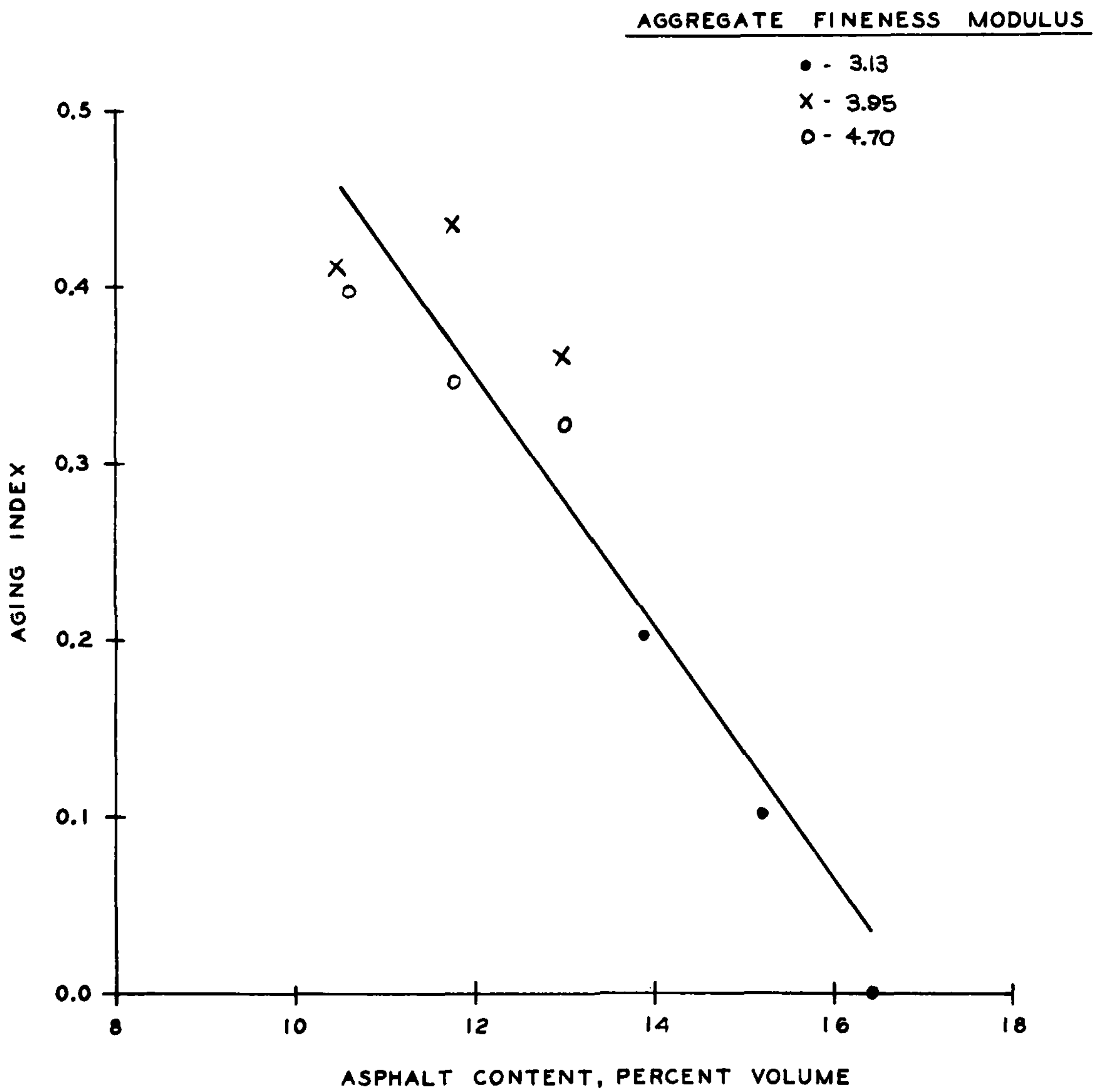


Figure B-42. Aging Relationship for Mixes of Limestone and S AC 20 Asphalt (225^{o}F, 7 Days)

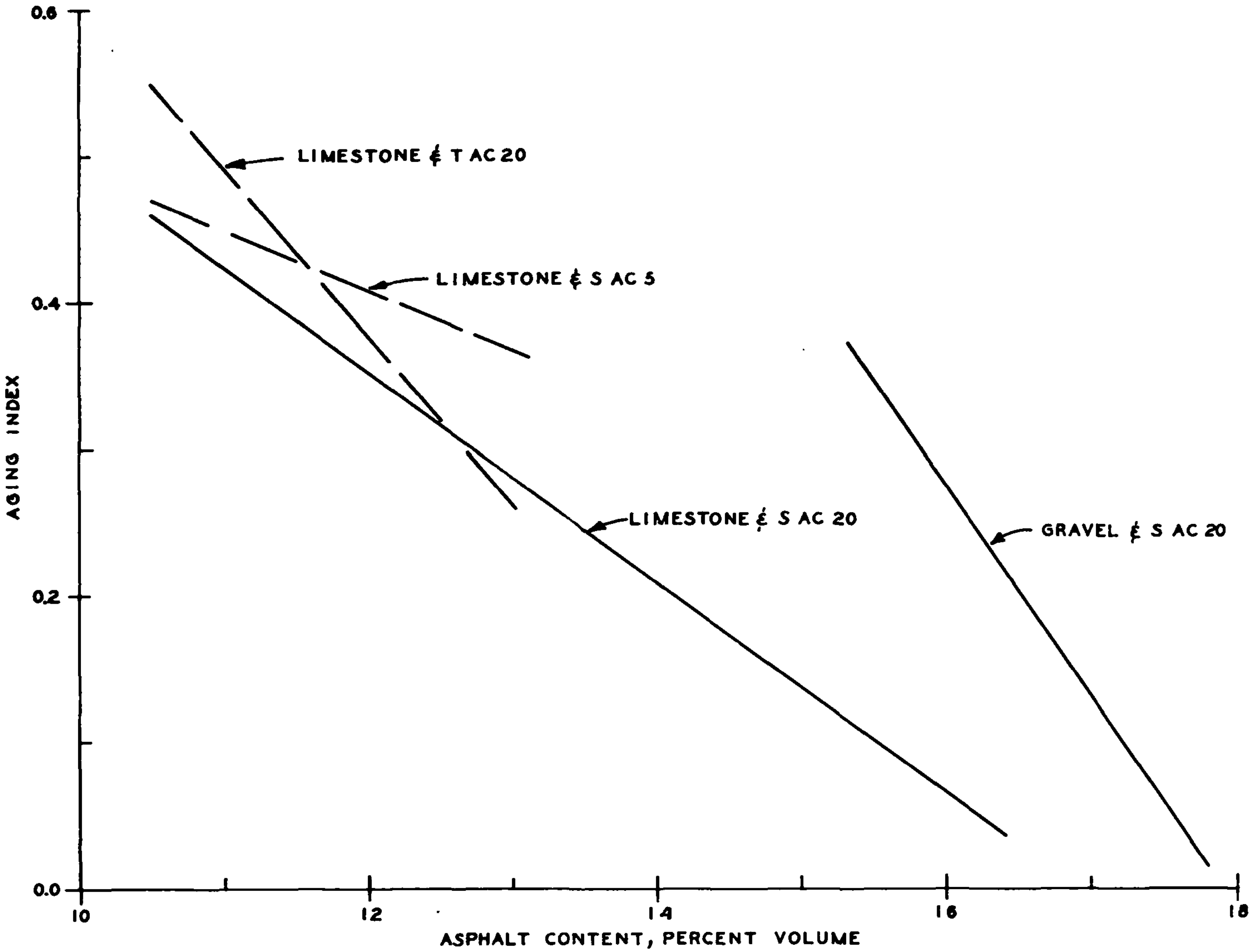


Figure B-43. Aging Relationships for All Mixes (225°F, 7 Days)

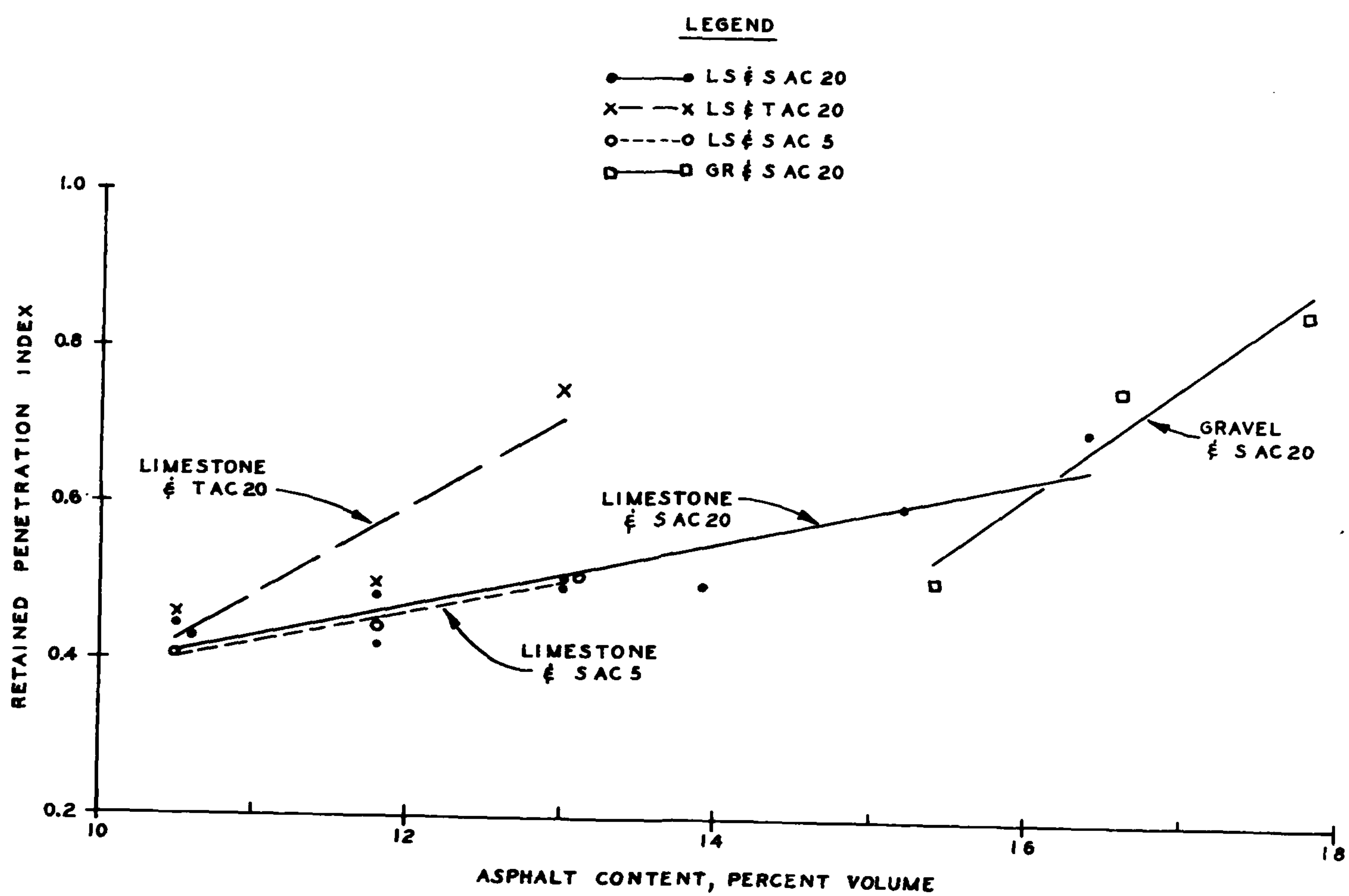


Figure B-44. Retained Penetration Index - Asphalt Content by Volume Relationships (225^{o}F, 7 Days)

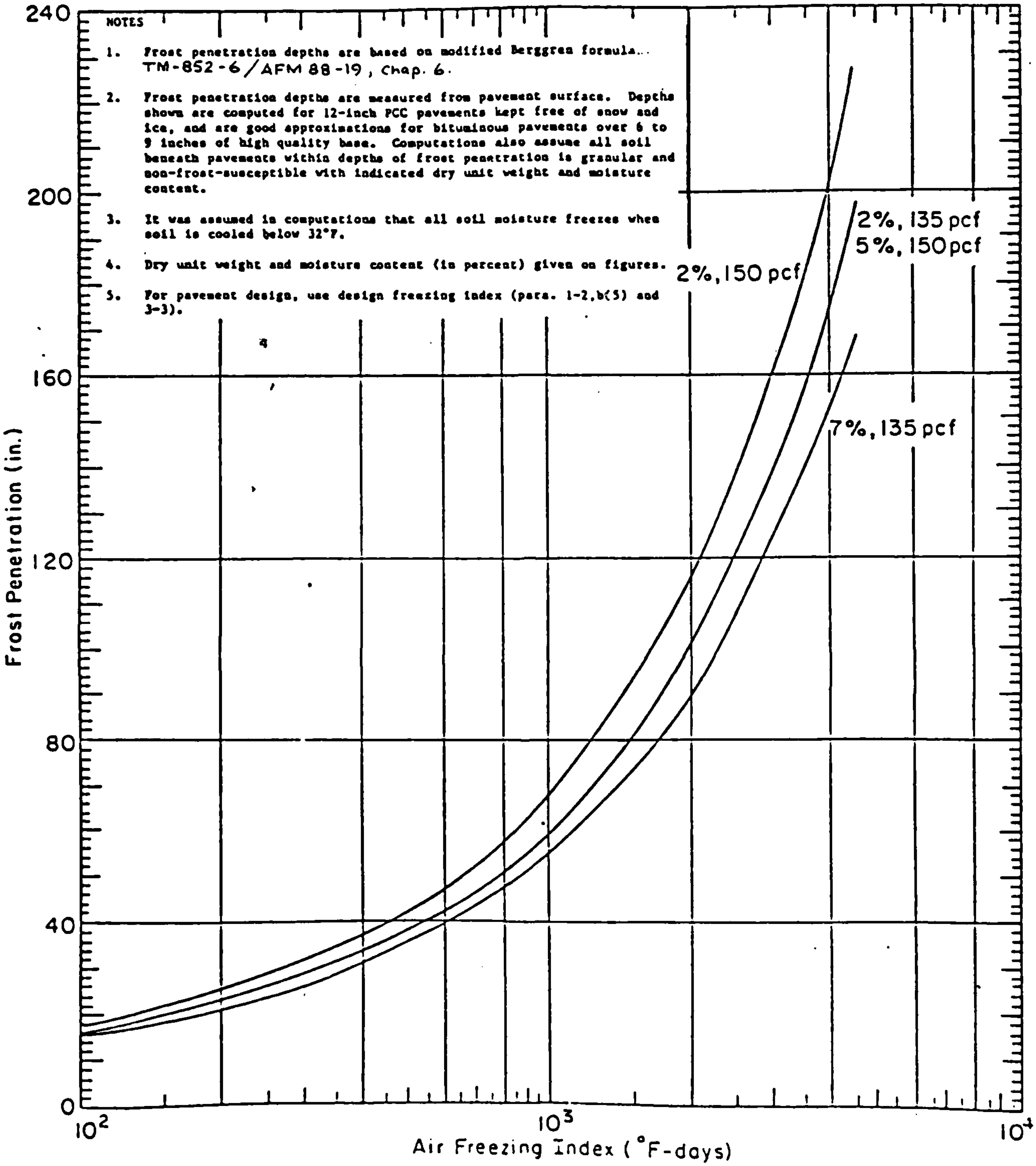


Figure B-45. Frost Penetration Beneath Pavements for Base Materials Having Densities of 150 and 135 pcf

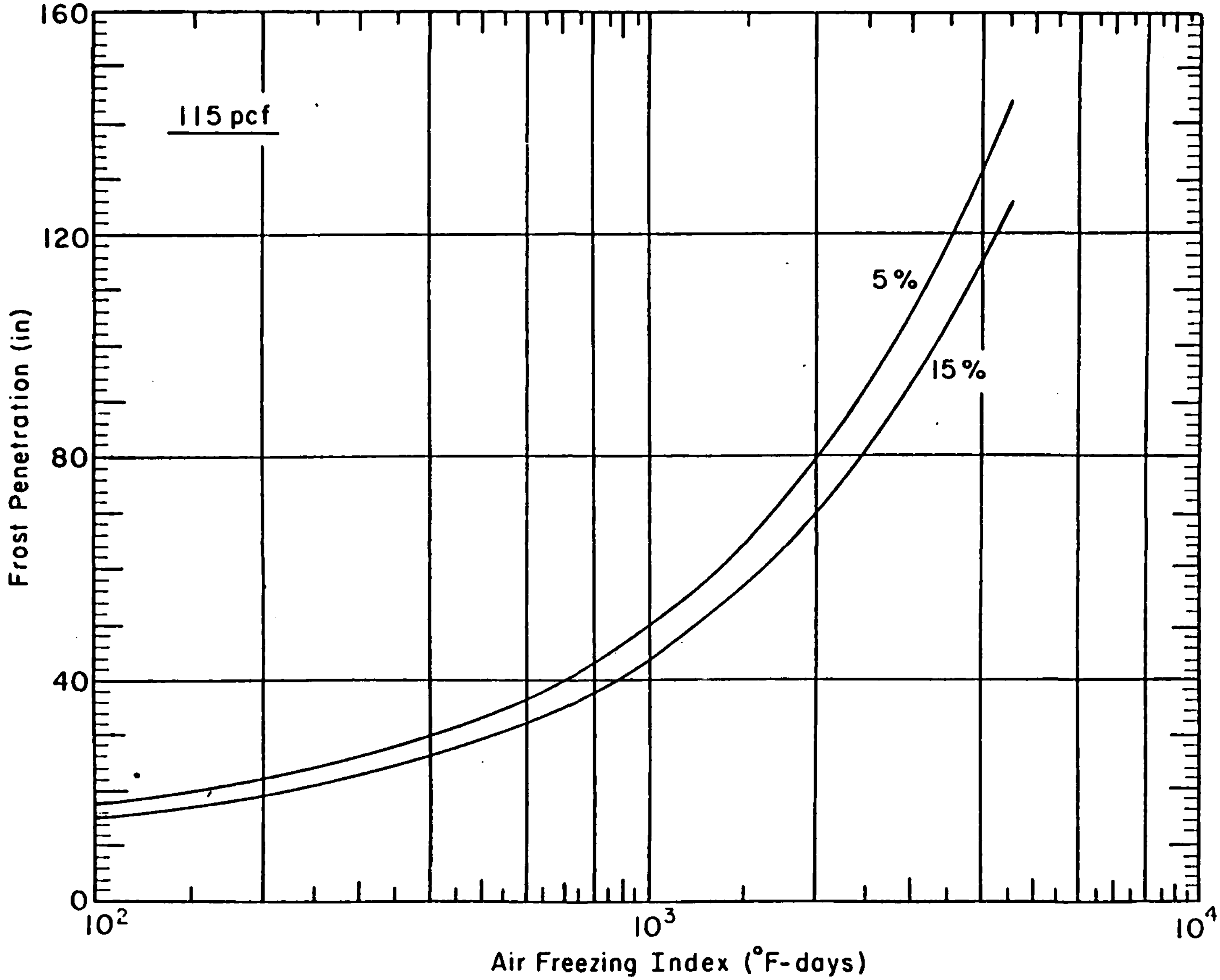
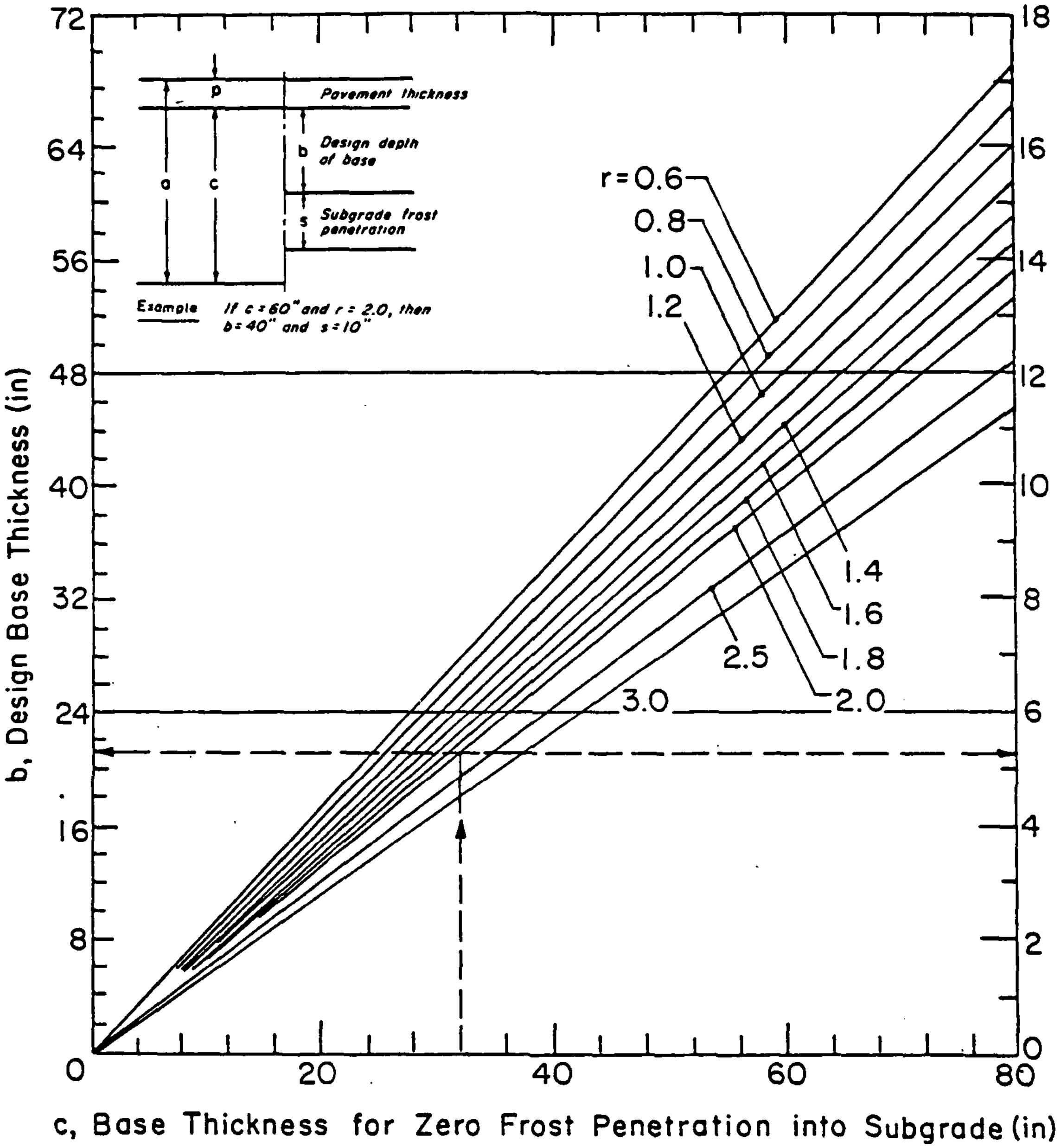


Figure B-46. Frost Penetration Beneath Pavements for Base Materials Having a Density of 115 pcf



NOTES

$\underline{a}$ = Combined thickness of pavement and non-frost-susceptible base for zero frost penetration into subgrade.

$\underline{c} = \underline{a} - \underline{p}$

$\underline{w}_b$ = Water content of base.

$\underline{w}_s$ = Water content of subgrade.

$\underline{r} = \frac{\underline{w}_s}{\underline{w}_b}$ Not to exceed 2.0 for type A and B areas on airfields and 3.0 for the other pavements.

Figure B-47. Design Depth of Non-Frost-Susceptible Base for Limited Subgrade Frost Penetration

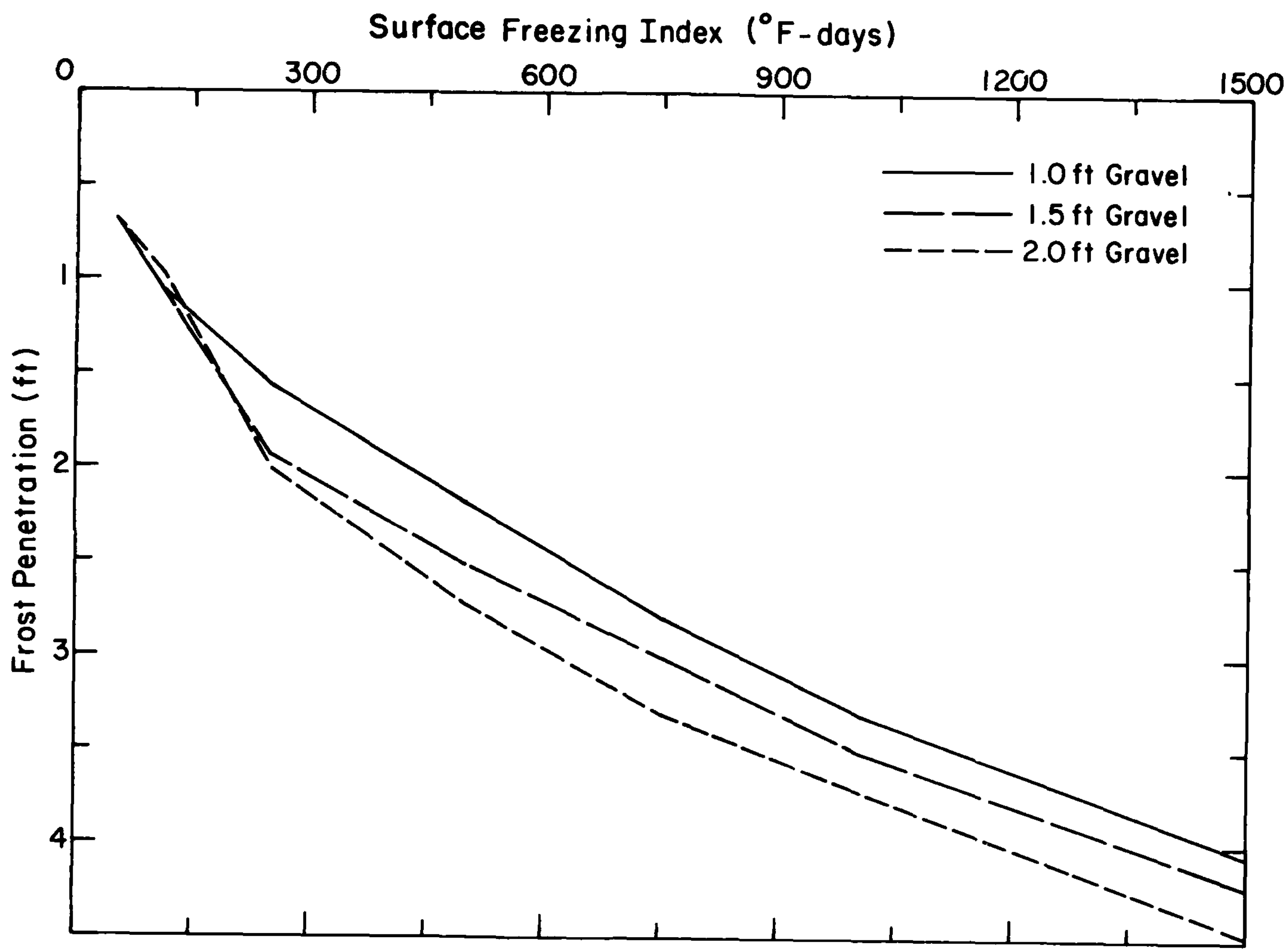


Figure B-48. Frost Penetration Beneath 3-inch Thick AC Pavements with Various Thicknesses of Gravel Base at 130 pcf and 5 Percent Moisture Content and Subgrade at 95 pcf and 18 Percent Moisture.

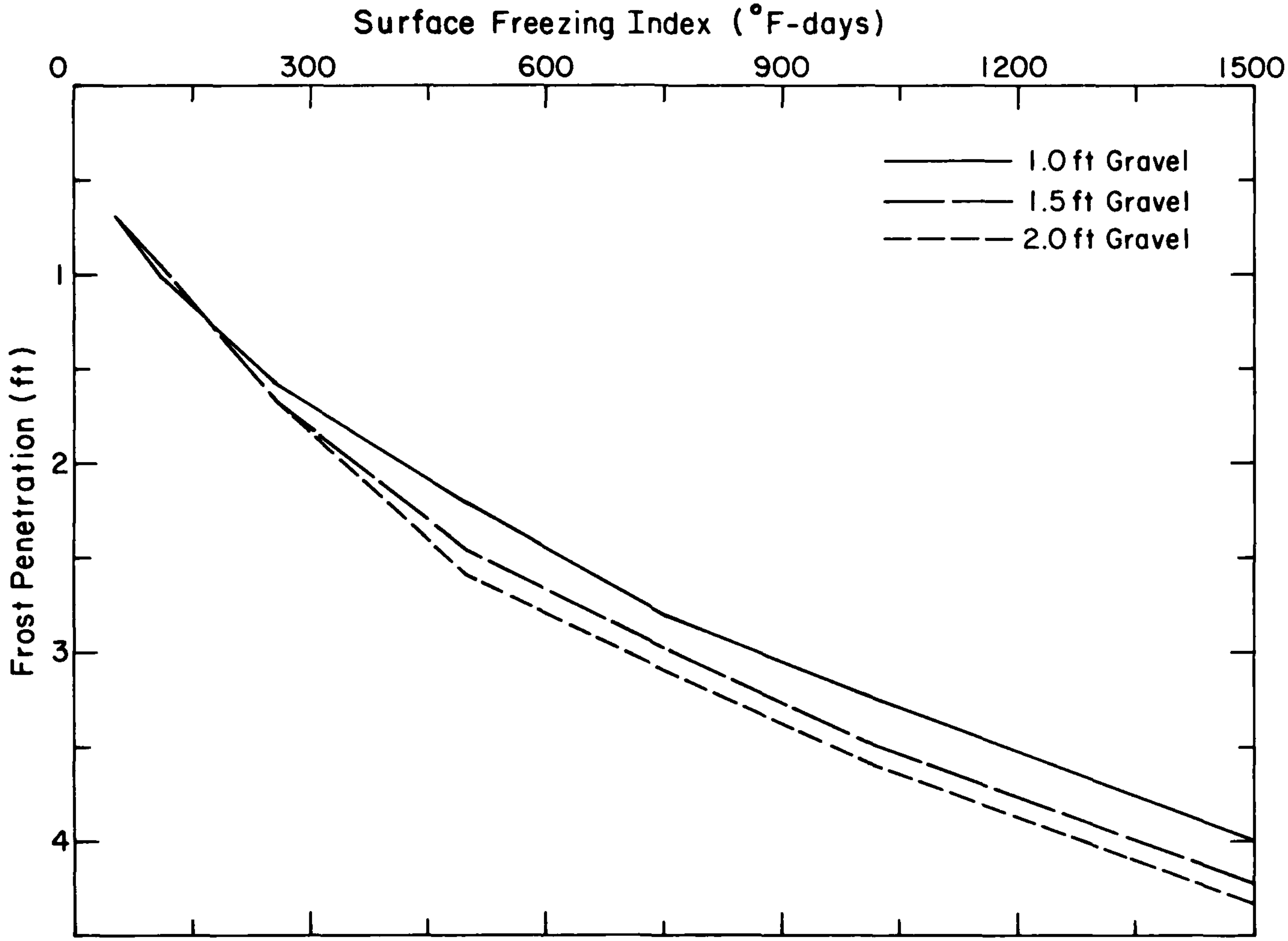


Figure B-49. Frost Penetration Beneath 3-inch Thick AC Pavements with Various Thicknesses of Gravel Base at 115 pcf and 8 Percent Moisture Content and Subgrade at 95 pcf and 18 Percent Moisture

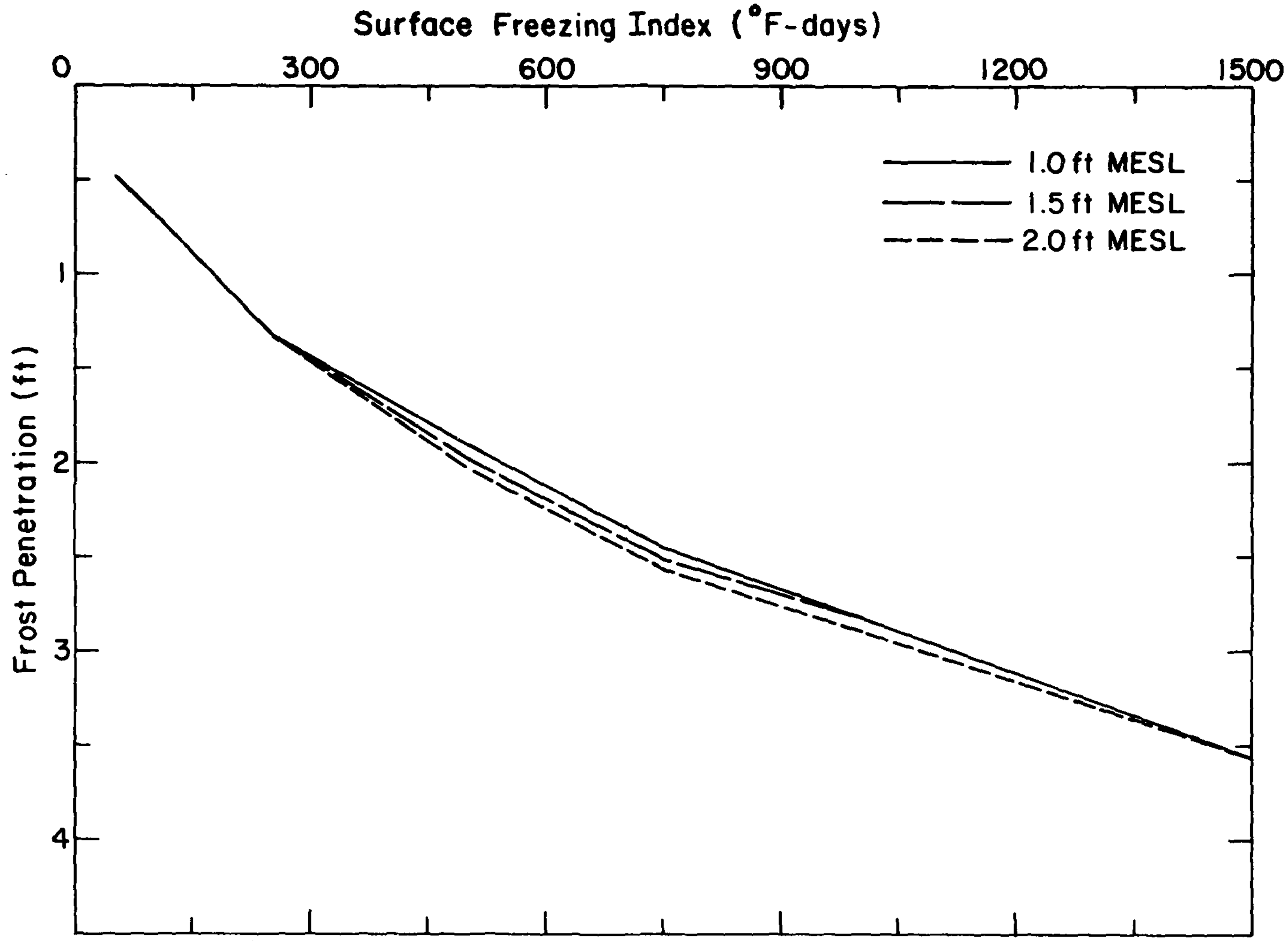


Figure B-50. Frost Penetration Beneath 3-inch Thick AC Pavements with Various Thicknesses of MESL (100 pcf at 12 Percent Moisture Content) Over a Subgrade at 95 pcf and 18 Percent Moisture Content

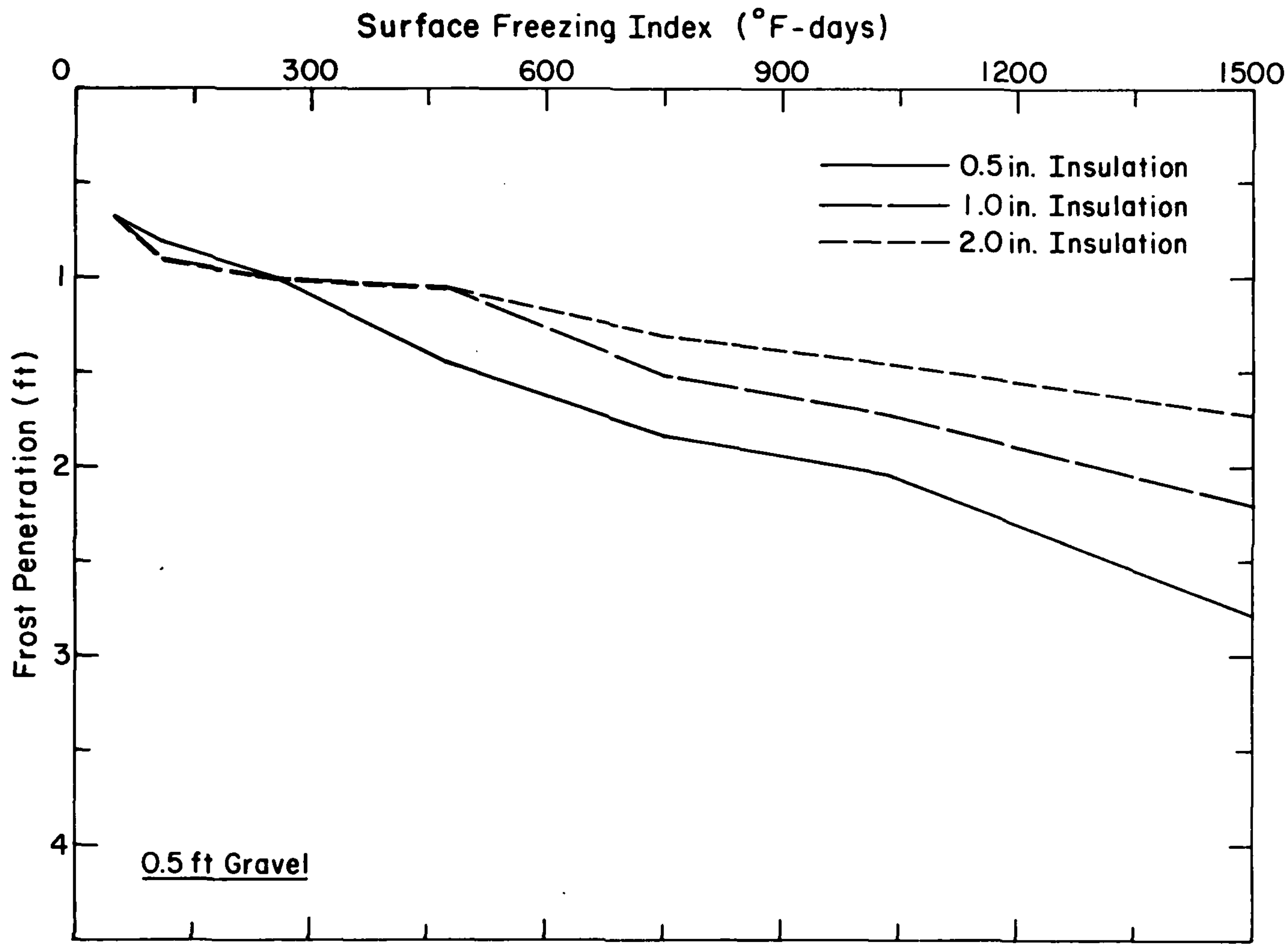


Figure B-51. Frost Penetration Beneath 3-inch Thick AC Pavements with 0.5 foot of Gravel (115 pcf at 8 Percent Moisture Content) and Extruded Polystyrene Insulation Over a Subgrade at 95 pcf and 18 Percent Moisture Content

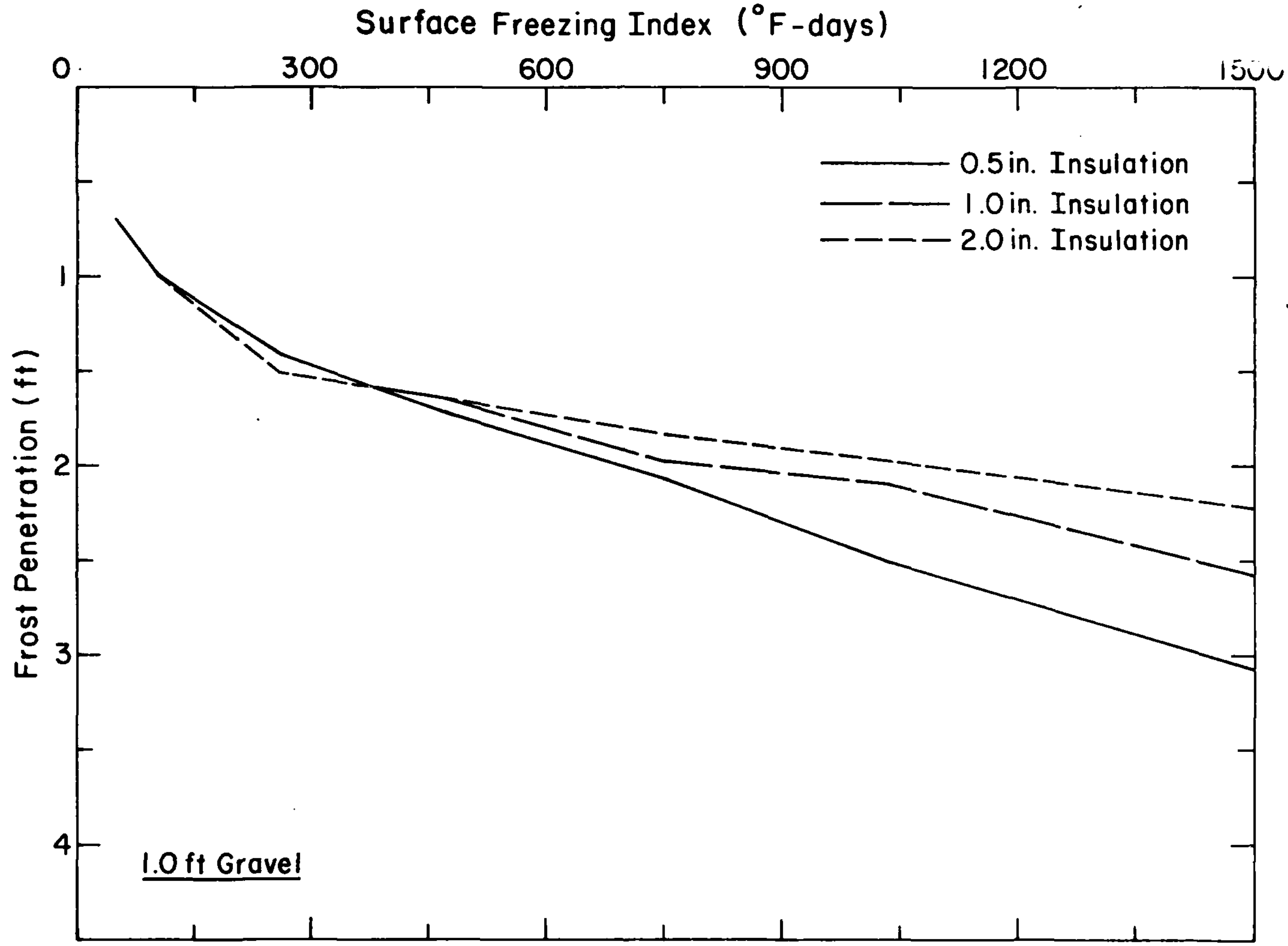


Figure B-52. Frost Penetration Beneath 3-inch Thick AC Pavements with 1 foot of gravel (115 pcf at 8 Percent Moisture Content) and Extruded Polystyrene Insulation Over a Subgrade at 95 pcf and 18 Percent Moisture Content

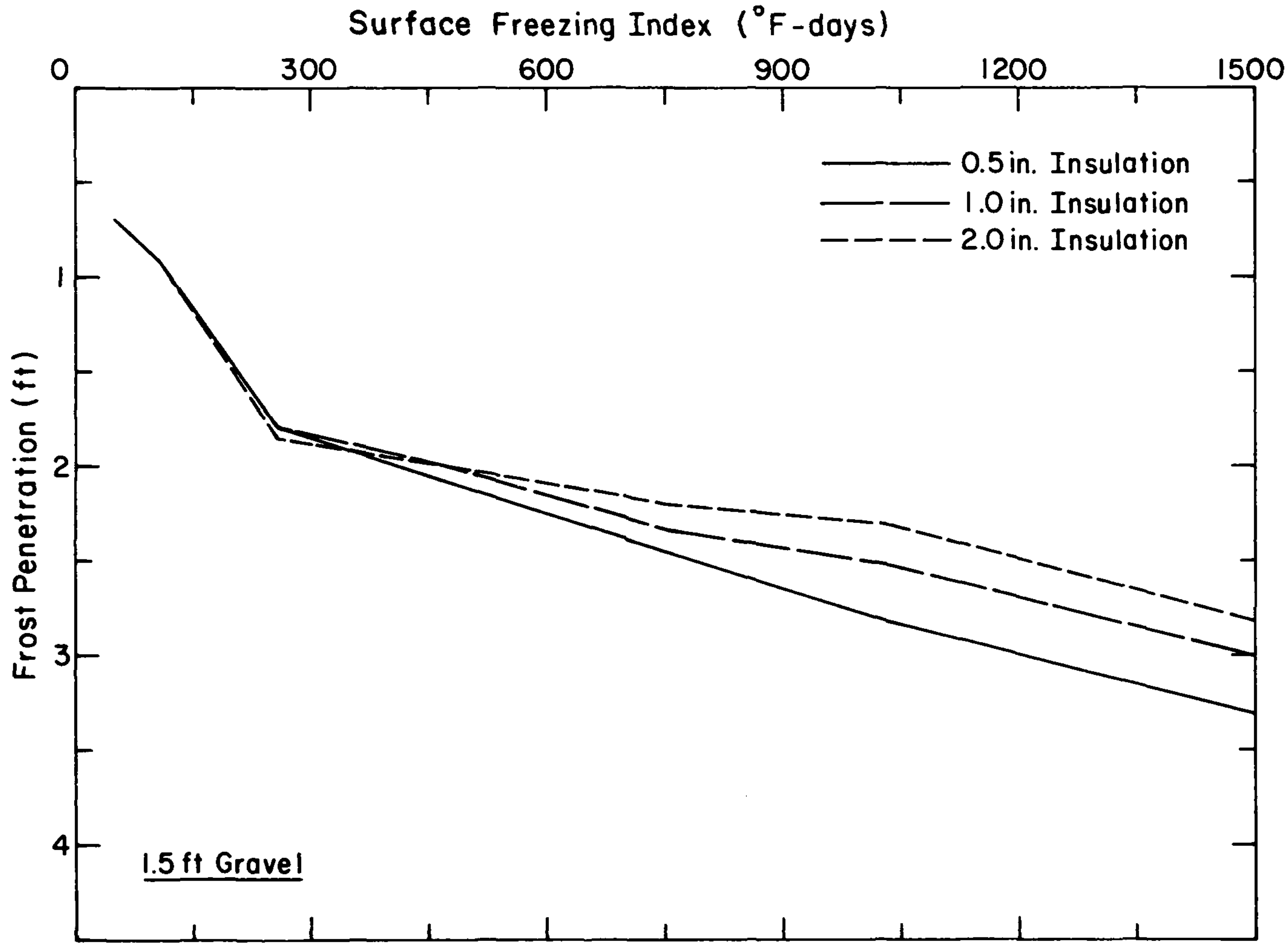


Figure B-53. Frost Penetration Beneath 3-inch Thick AC Pavements with 1.5 feet of gravel (115 pcf at 8 Percent Moisture Content) and Extruded Polystyrene Insulation Over a Subgrade at 95 pcf and 18 Percent Moisture Content

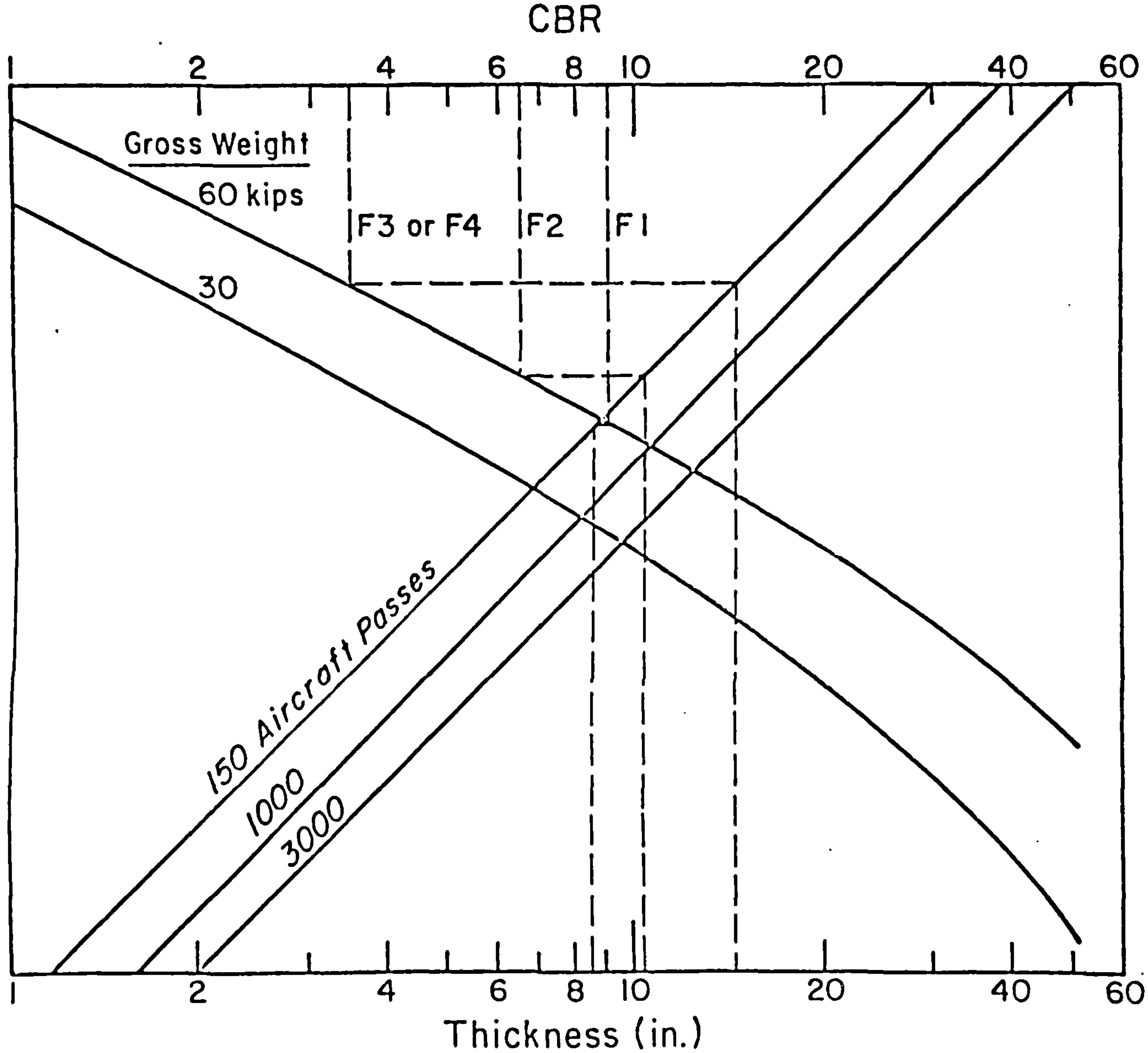


Figure B-54. Flexible Pavement Design Curves for ALRS Pavements

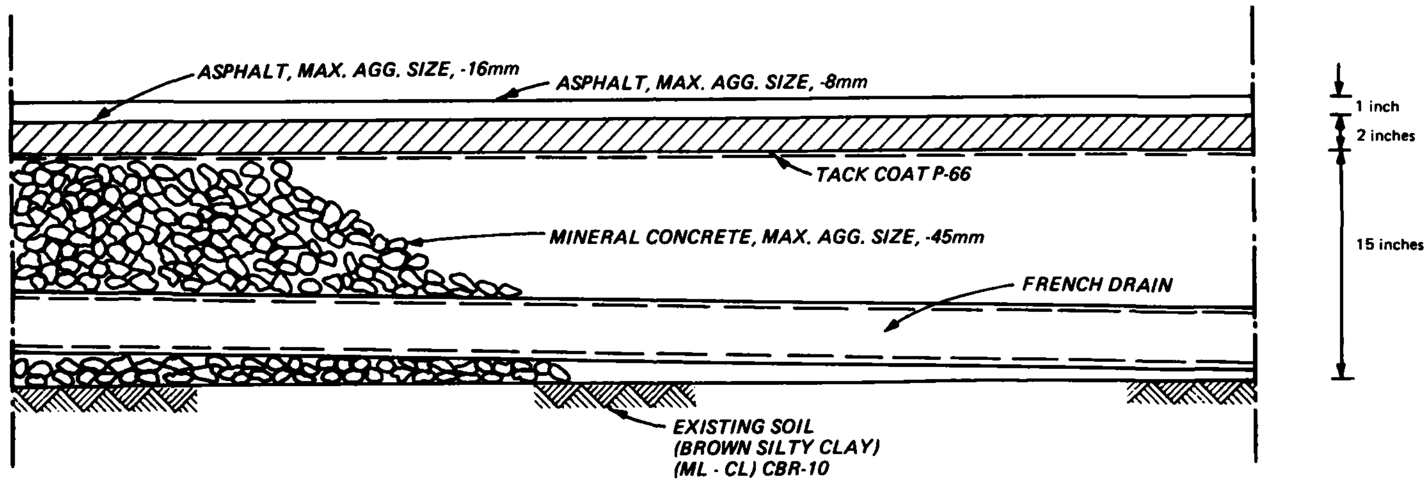


Figure B-55. Design Pavement Structure for Hahn ALRS

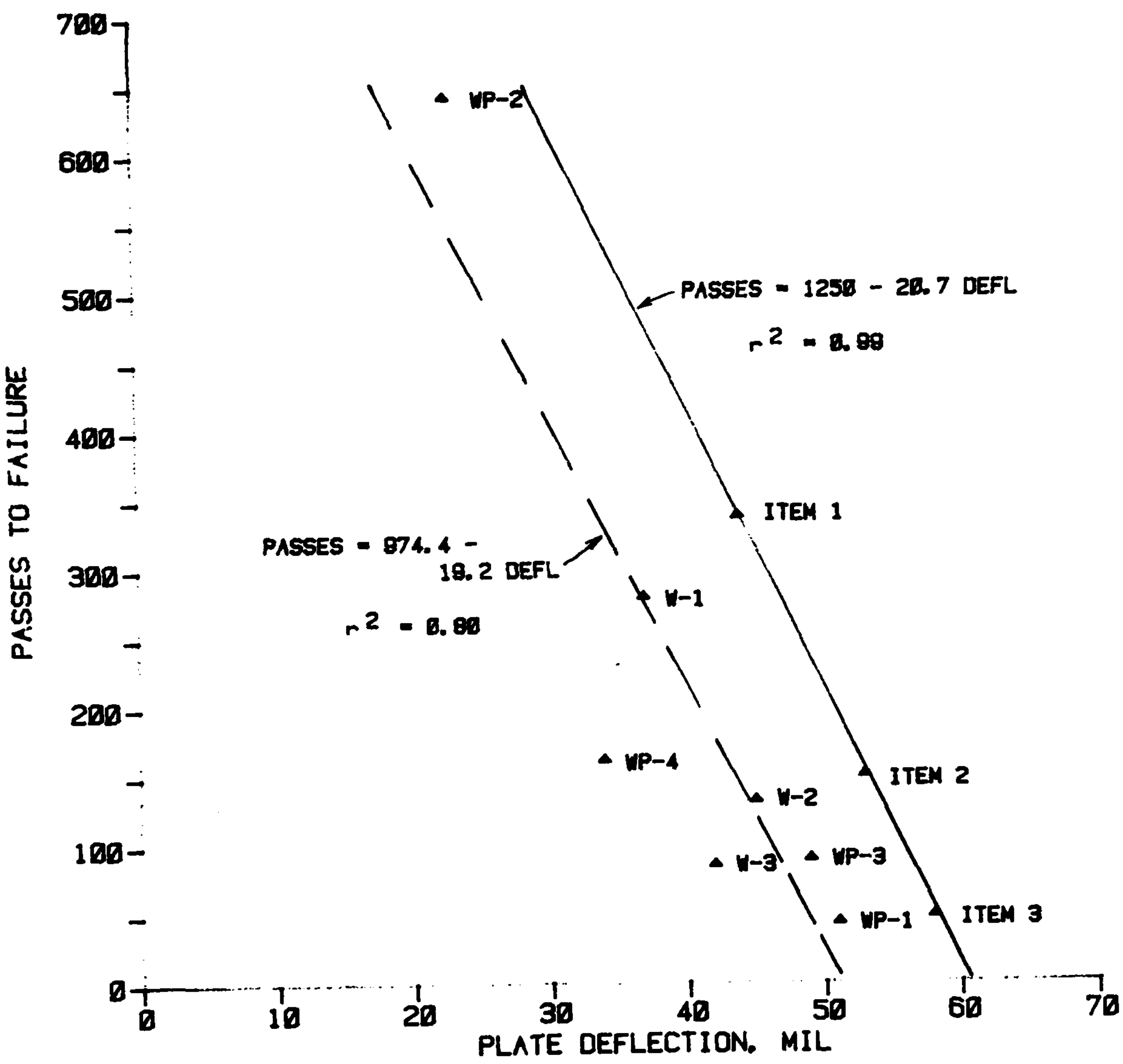


Figure B-56. Falling-Weight Deflectometer Deflections Compared to Passes to Failure

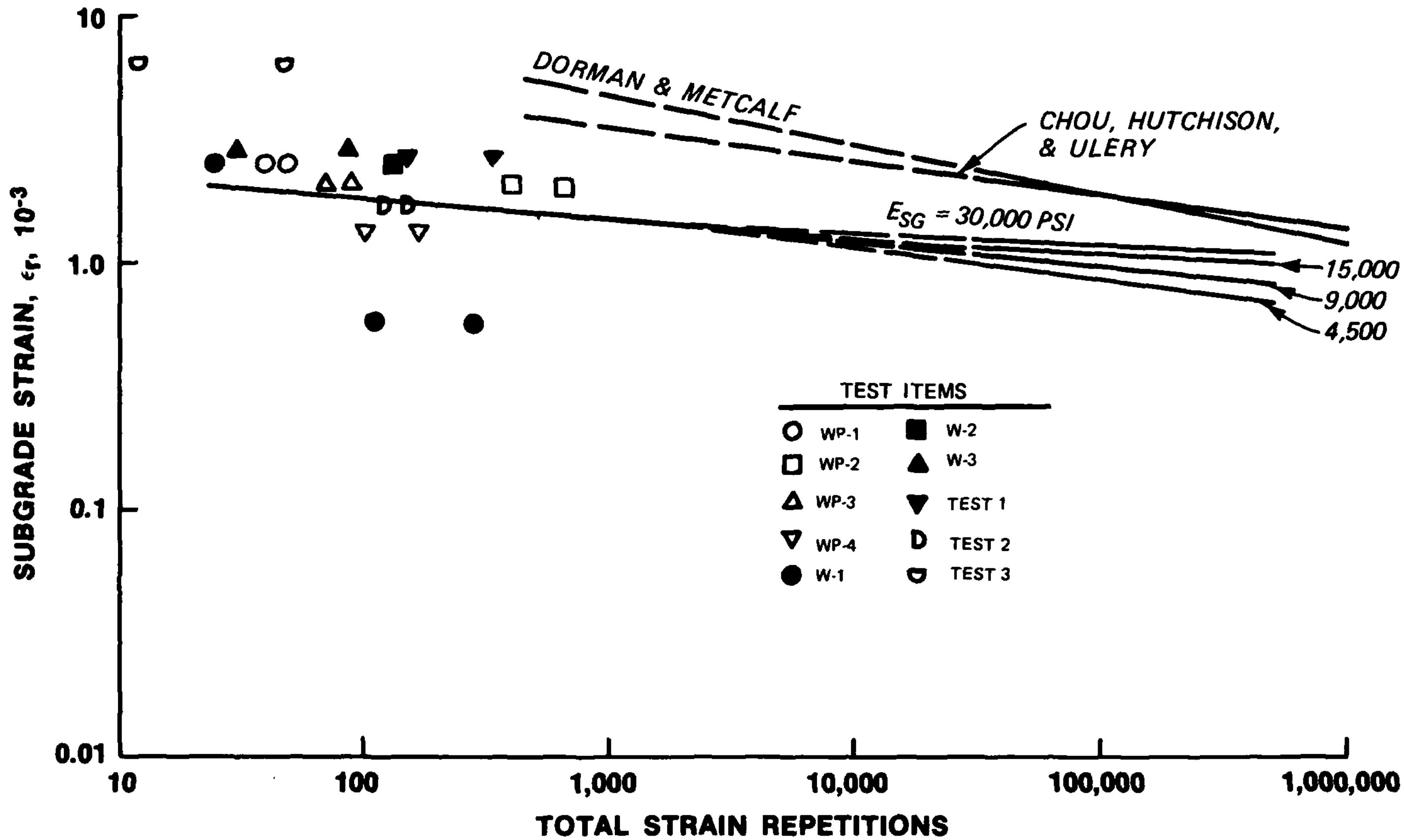


Figure B-57. Limiting Vertical Strain in the Subgrade

APPENDIX C

PHOTOGRAPHS

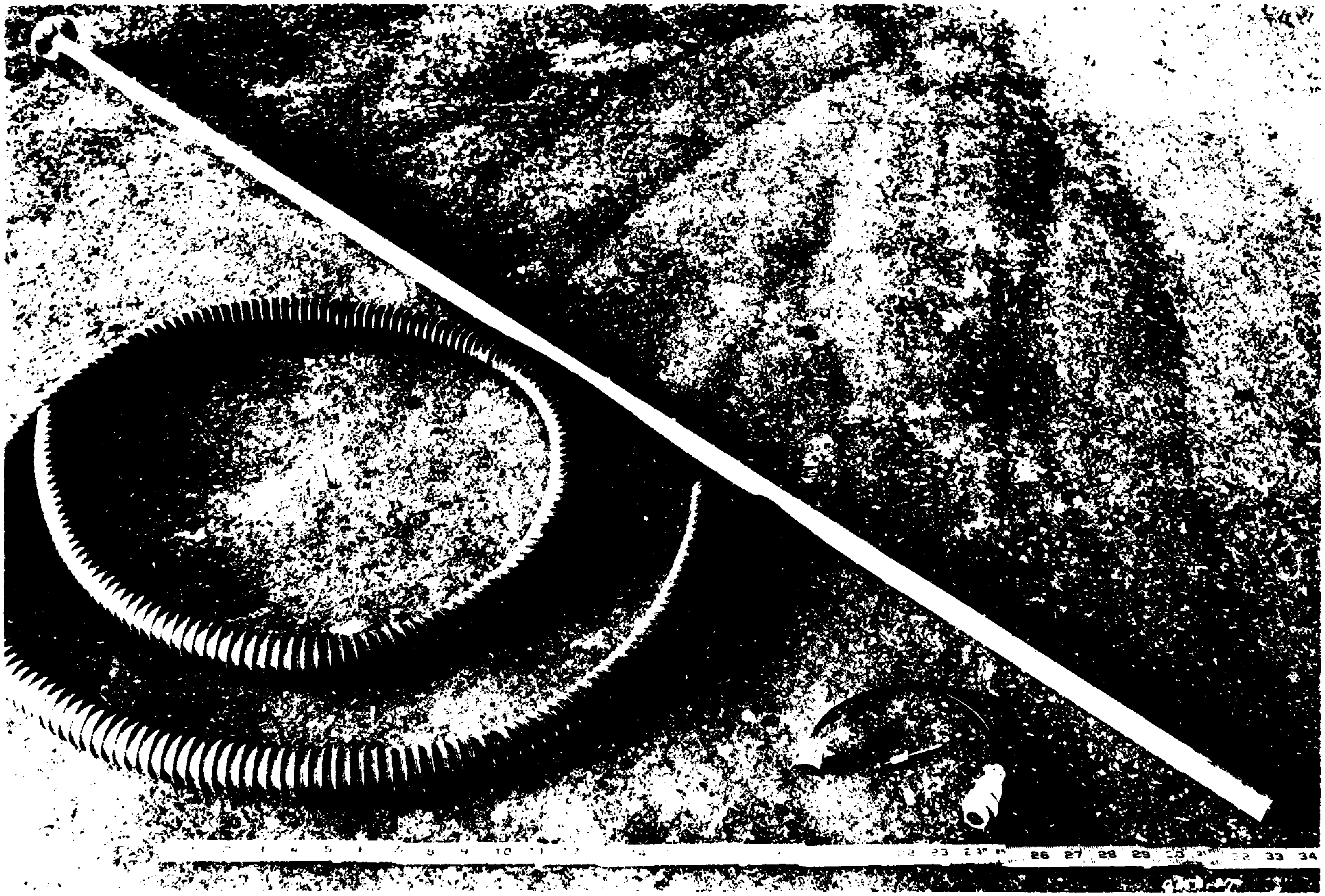


PHOTO C-1. Displacement Gage, Reference Rod and Flexible Casing

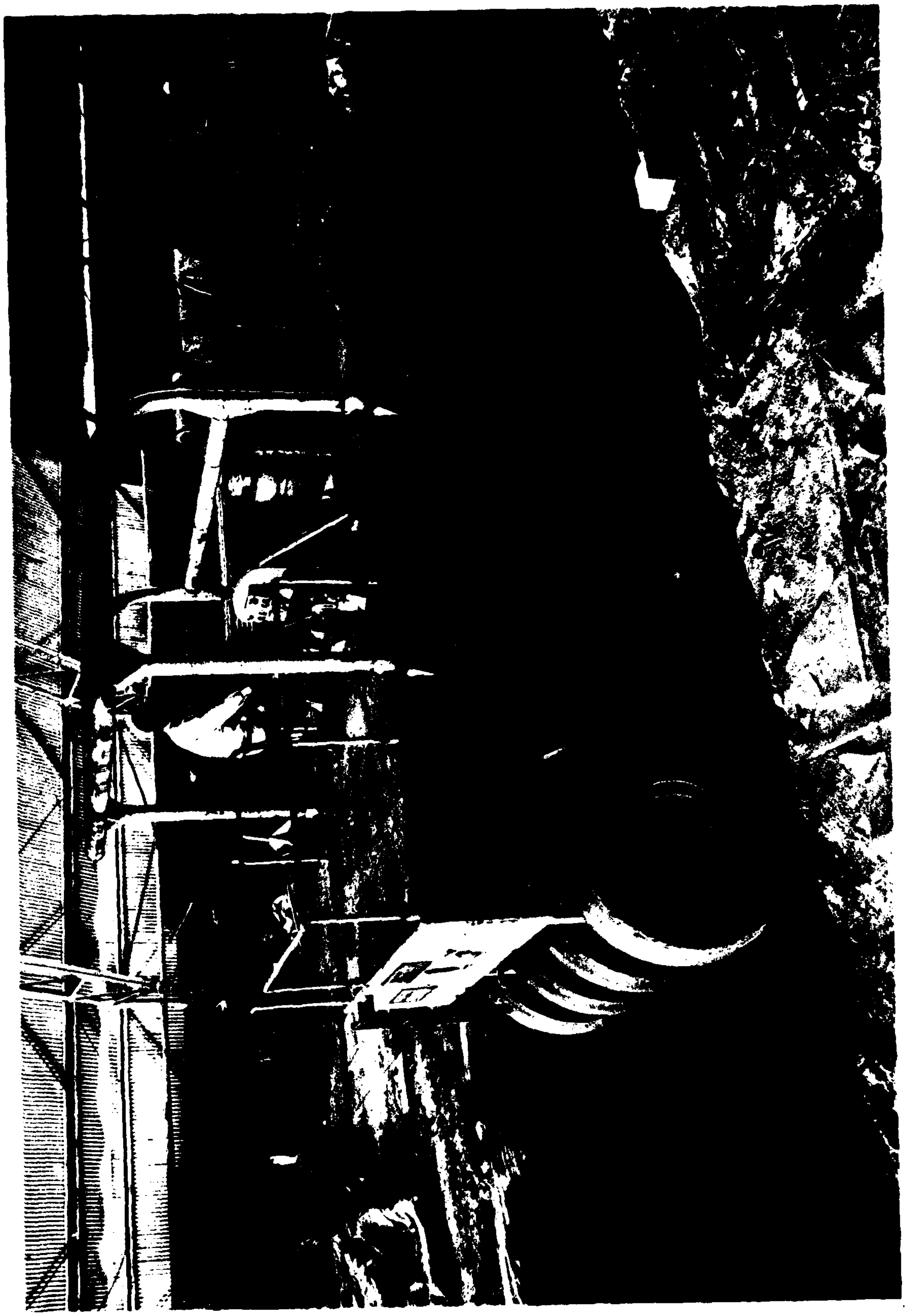


PHOTO C-2. Compacting Clay Subgrade with 50-kip Roller

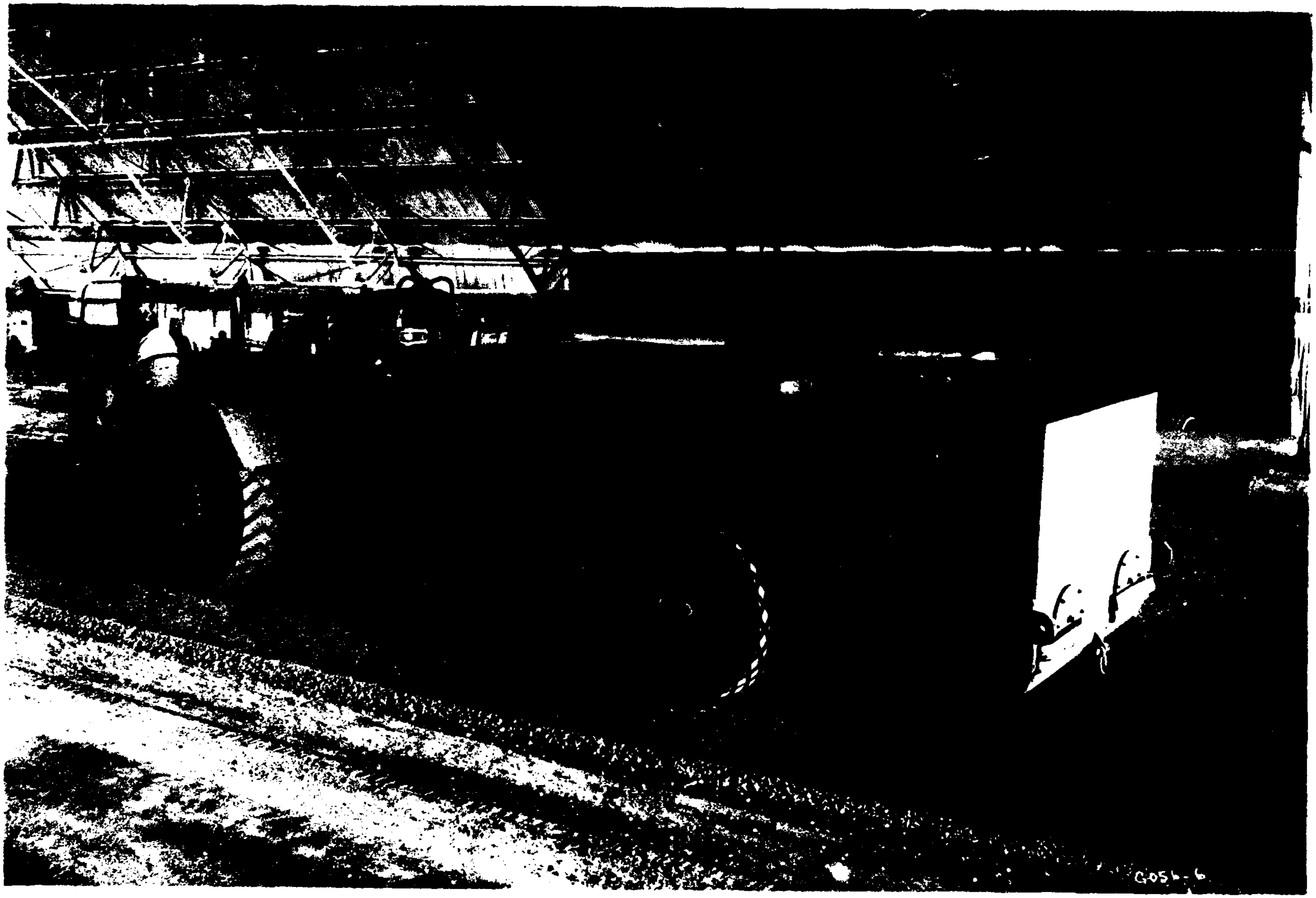


PHOTO C-3. Compacting Limestone with 50-ton Roller

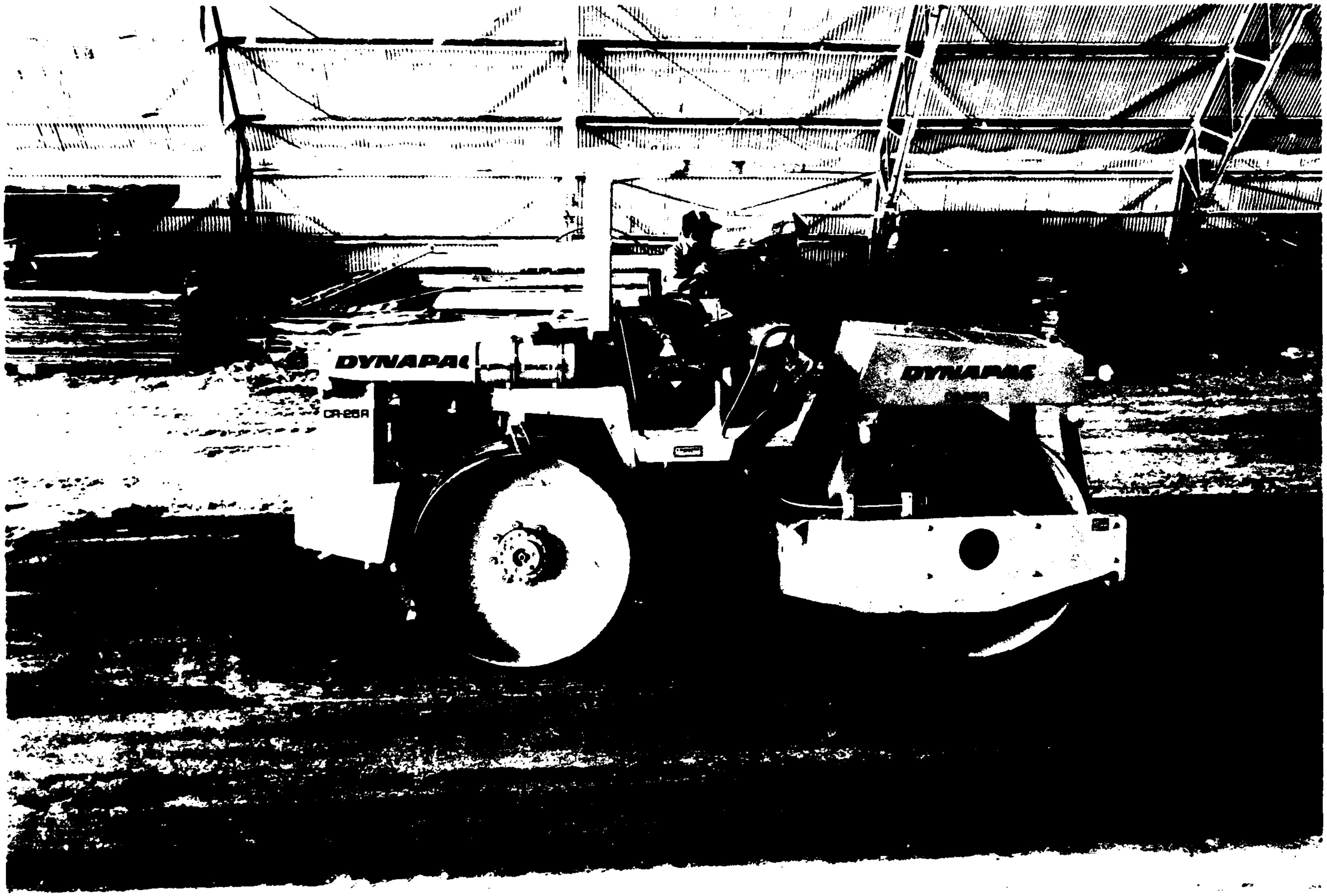


PHOTO C-4. Compacting Asphalt with Vibratory Roller

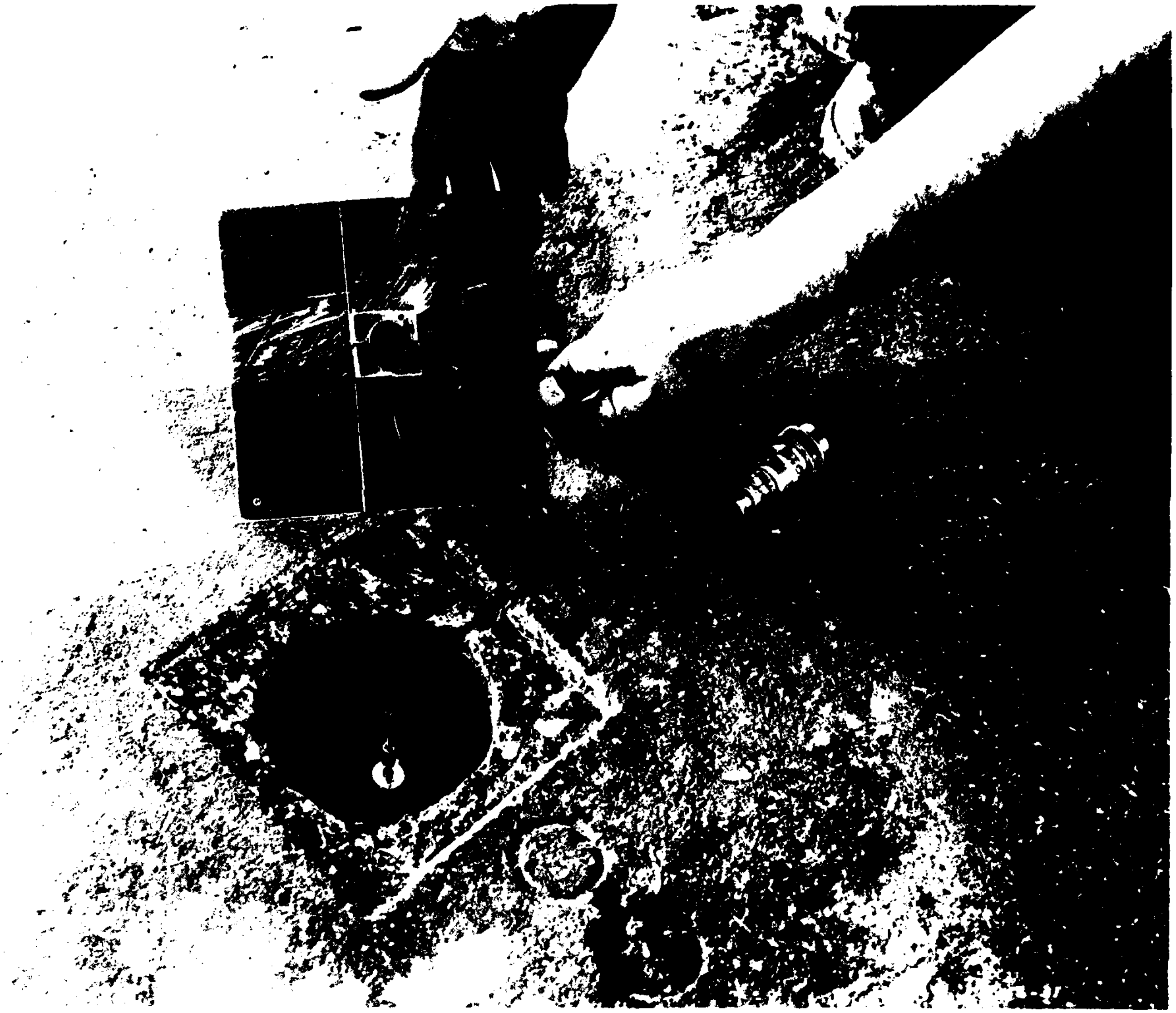


PHOTO C-5. Displacement Gage Installation Prior to Placing Surface Plate

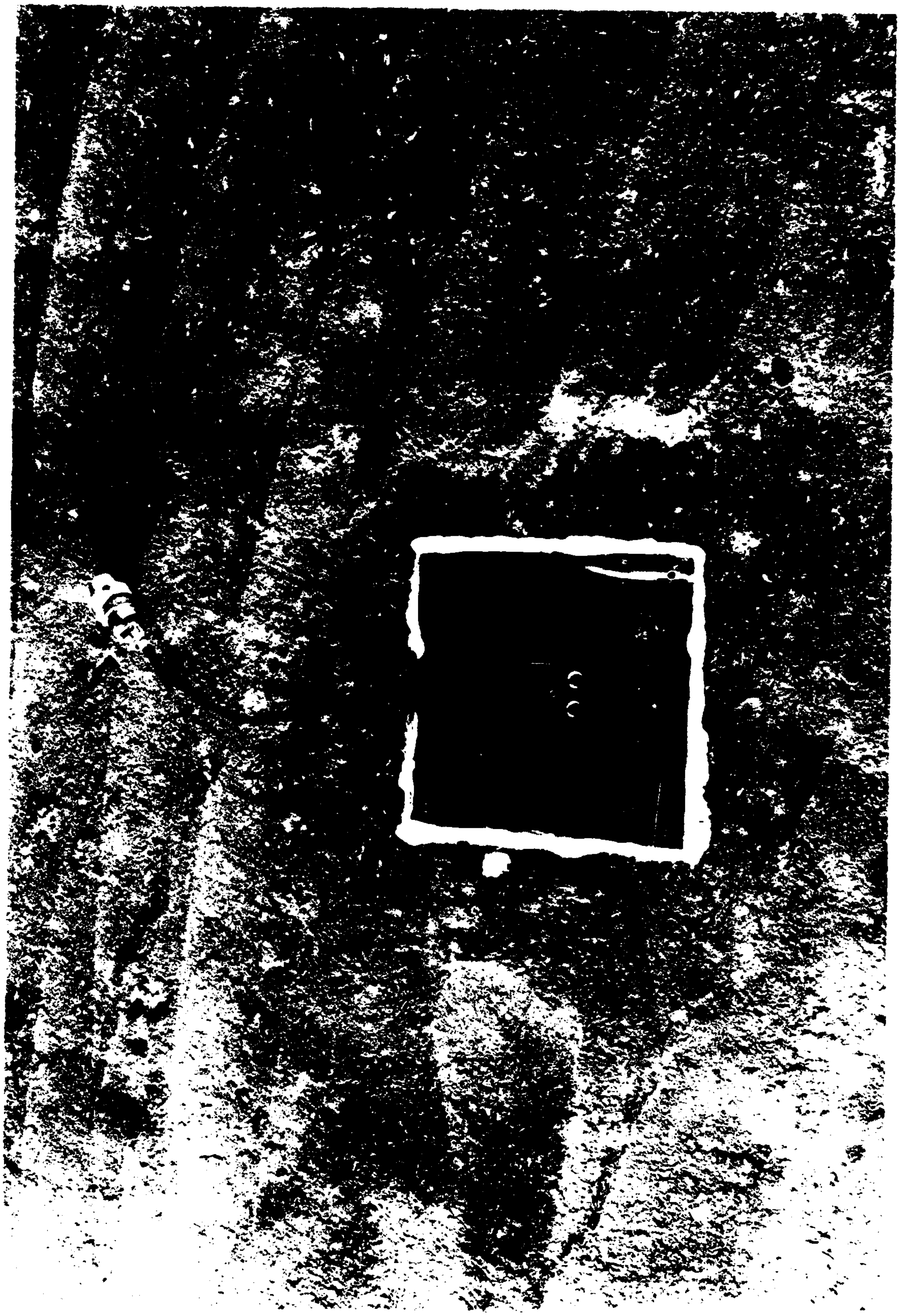


PHOTO C-6. Displacement Gage After Installation

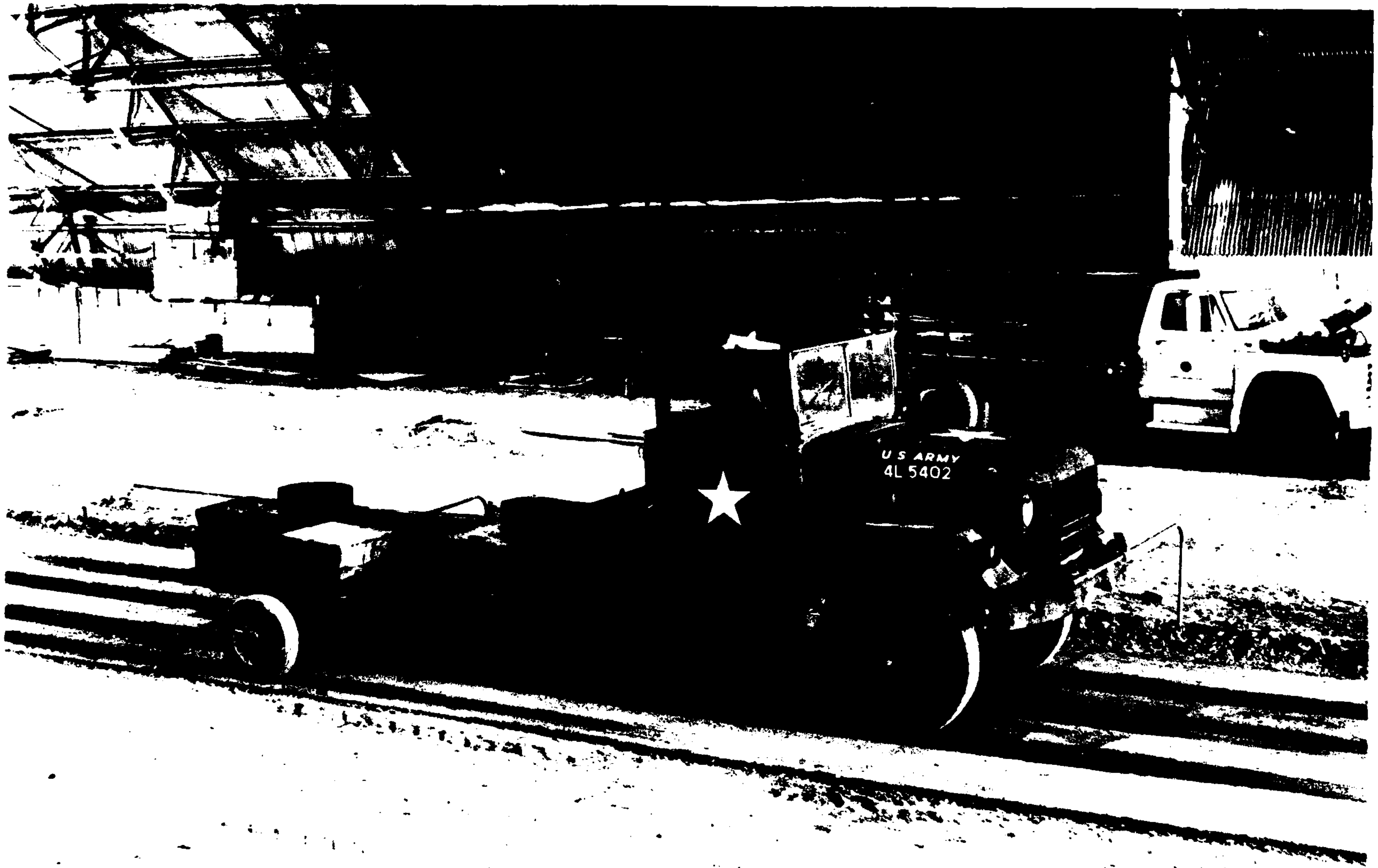


PHOTO C-7. F-4 Aircraft Load Cart

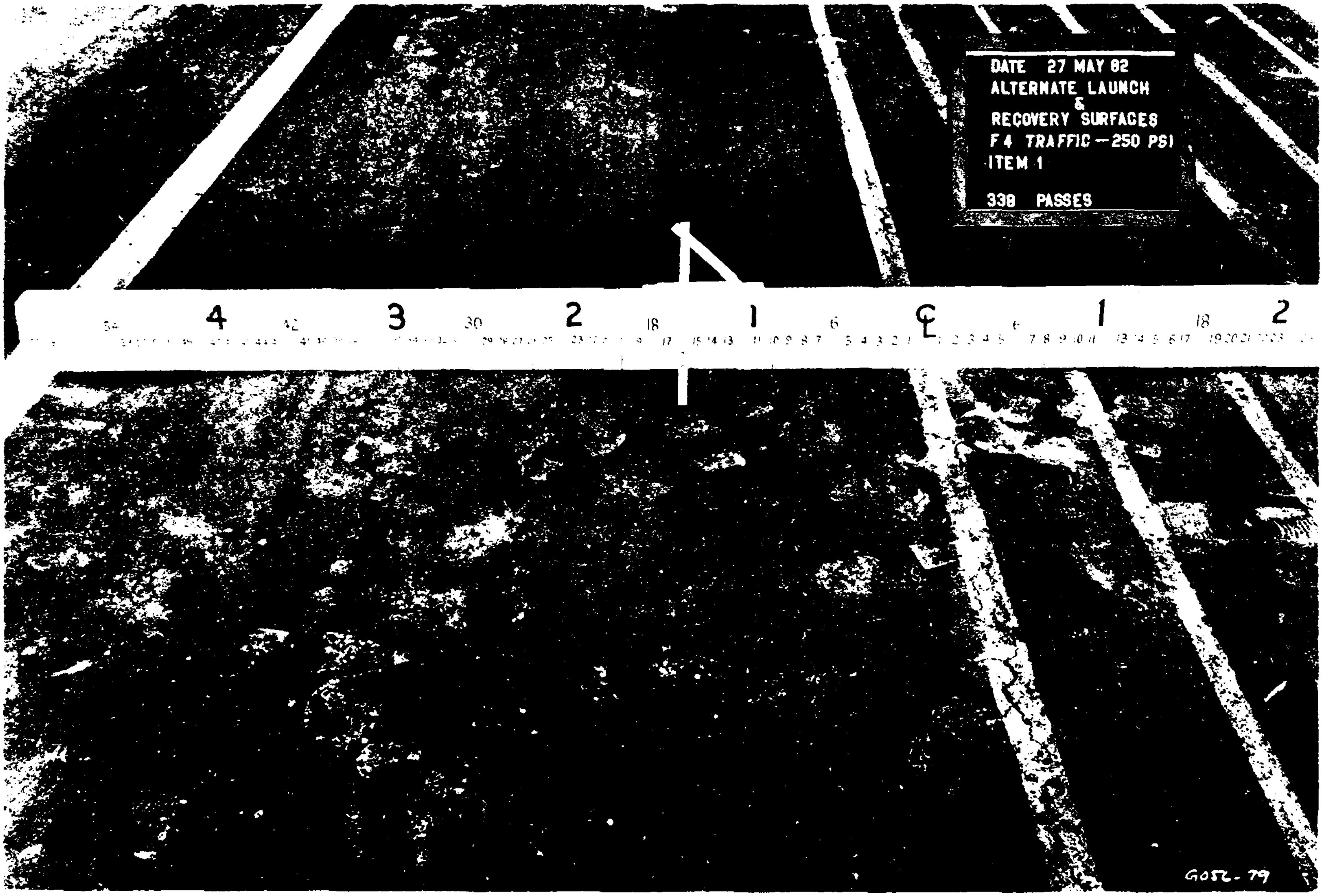


PHOTO C-8. Failure of 2-inch Asphalt (Item 1) Under Distributed Traffic

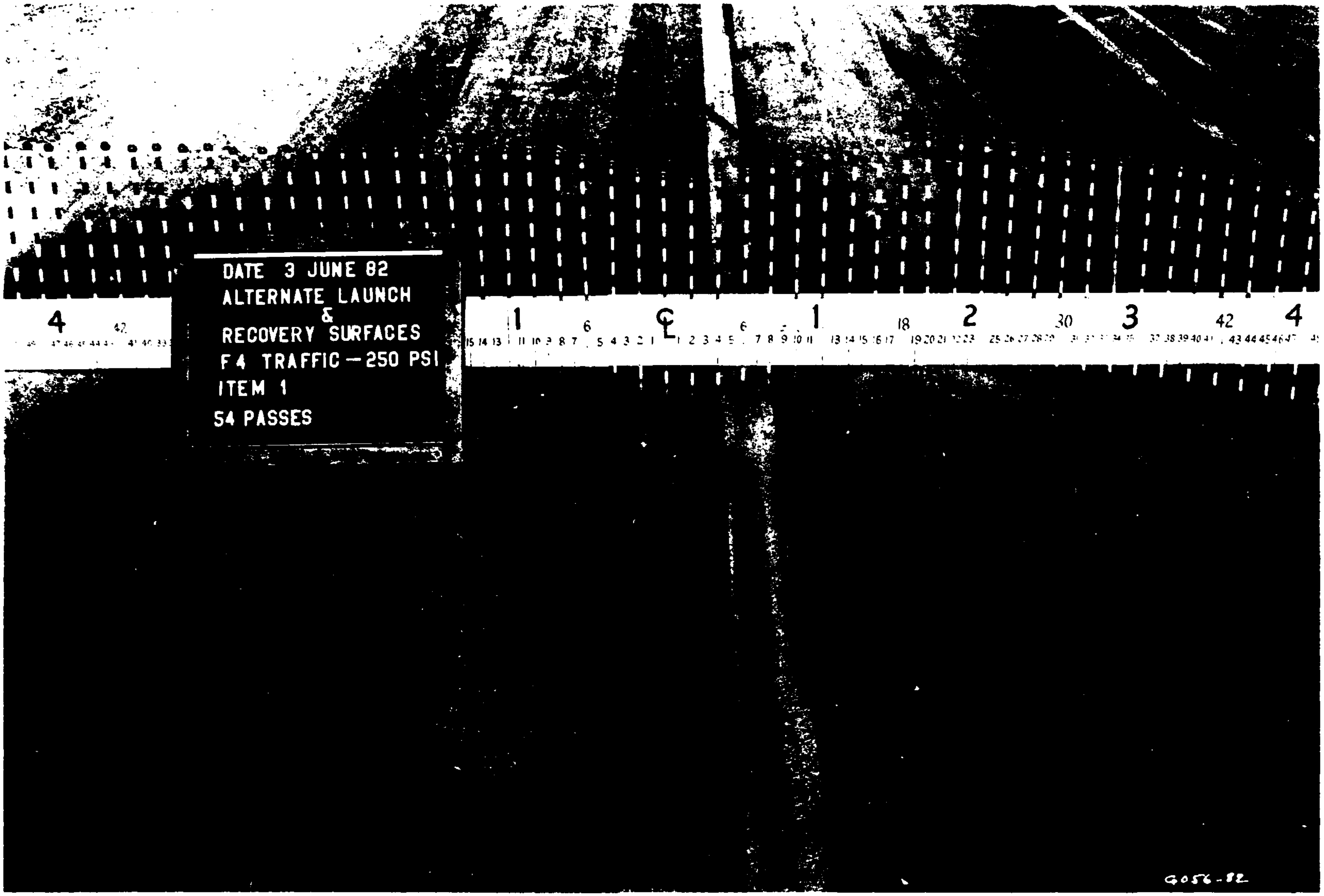


PHOTO C-9. Failure of 2-inch Asphalt (Item 1) Under Channelized Traffic

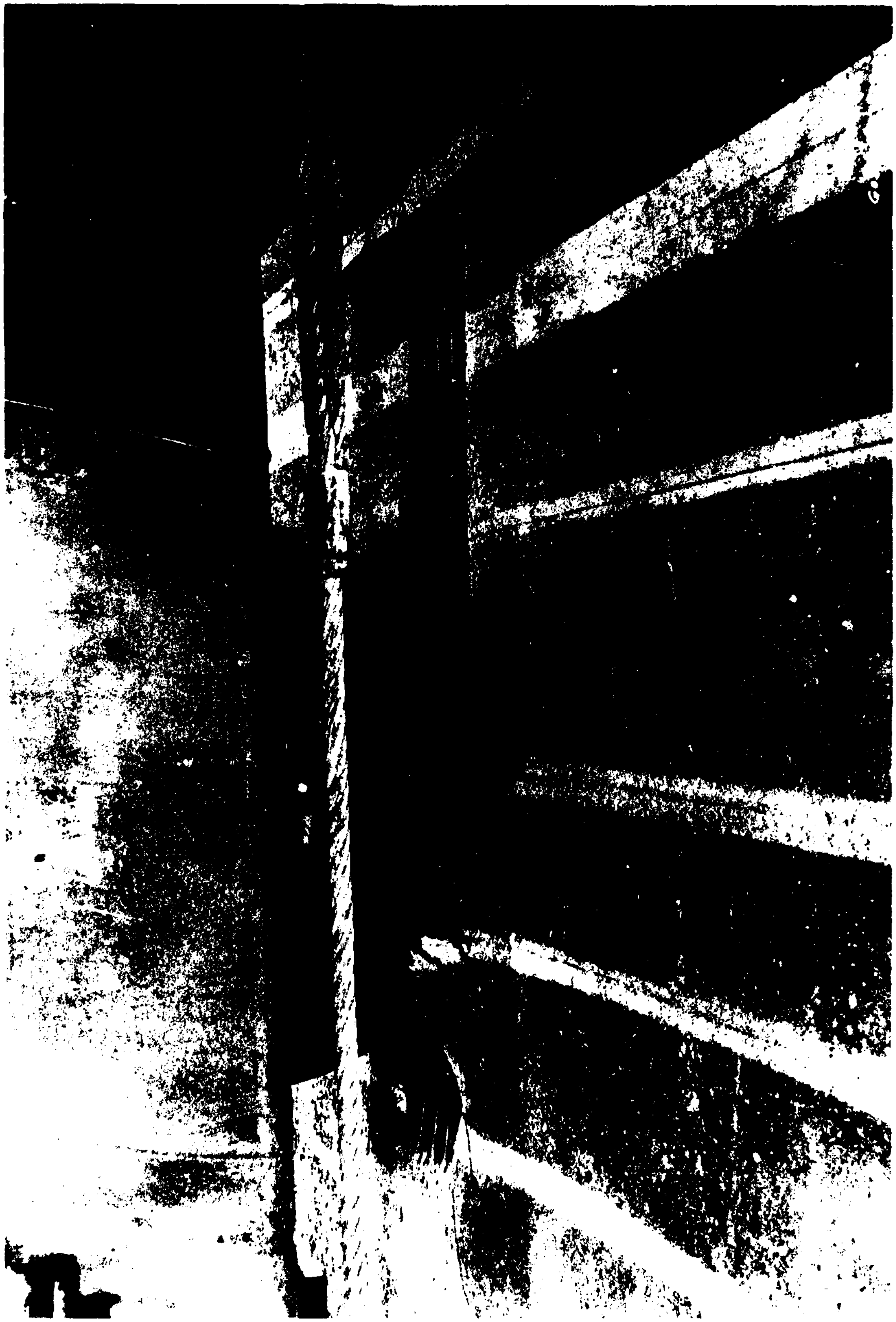


PHOTO C-10. Failure of 2-inch Asphalt (Item 1) After Four Locked-Wheel Skids

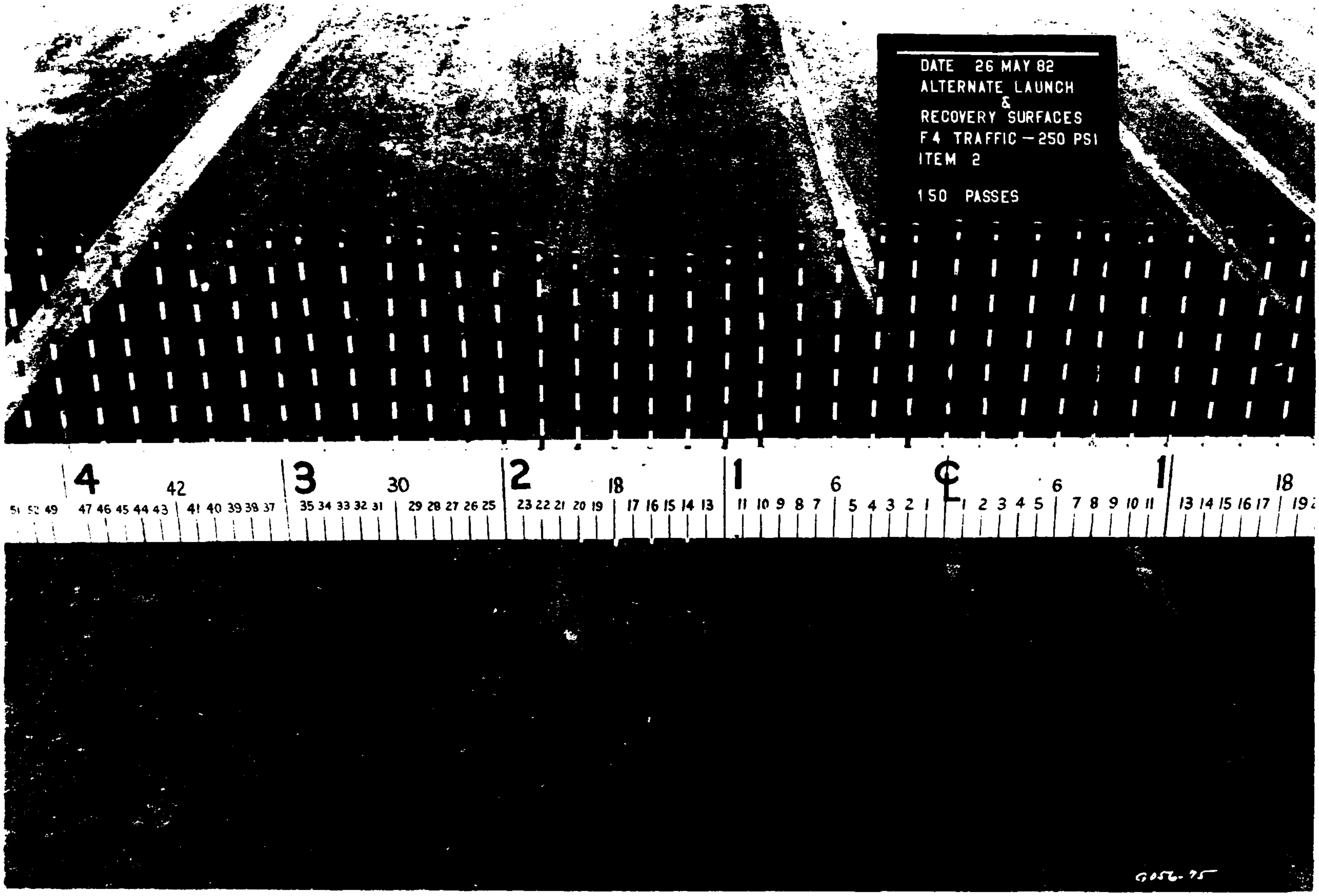


PHOTO C-11. Failure of 1-inch Asphalt (Item 2) Under Distributed Traffic

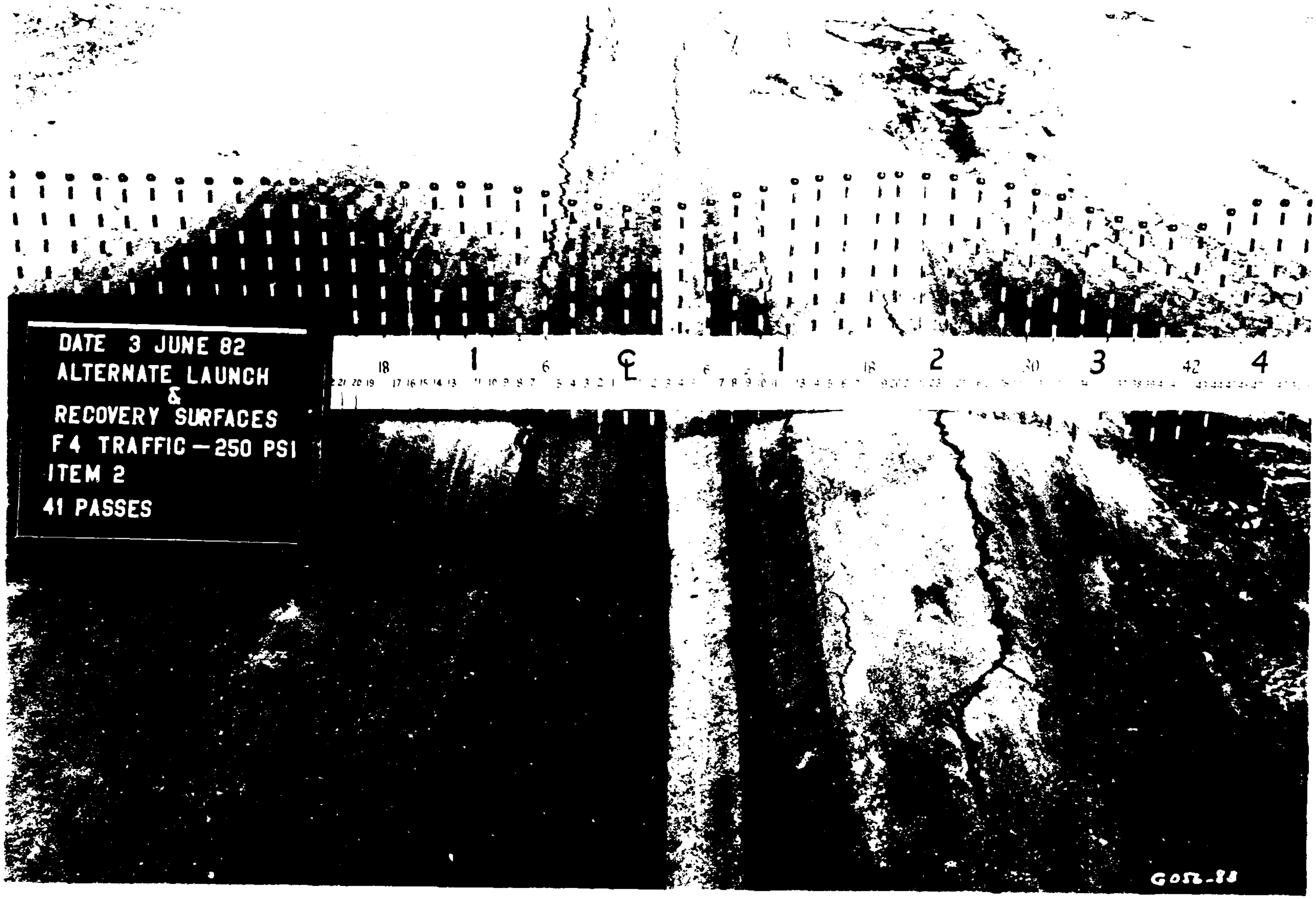


PHOTO C-12. Failure of 1-inch Asphalt (Item 2) Under Channelized Traffic

PHOTO C-13. Failure of 1-inch Asphalt (Item 2) After Two Locked-Wheel Skids

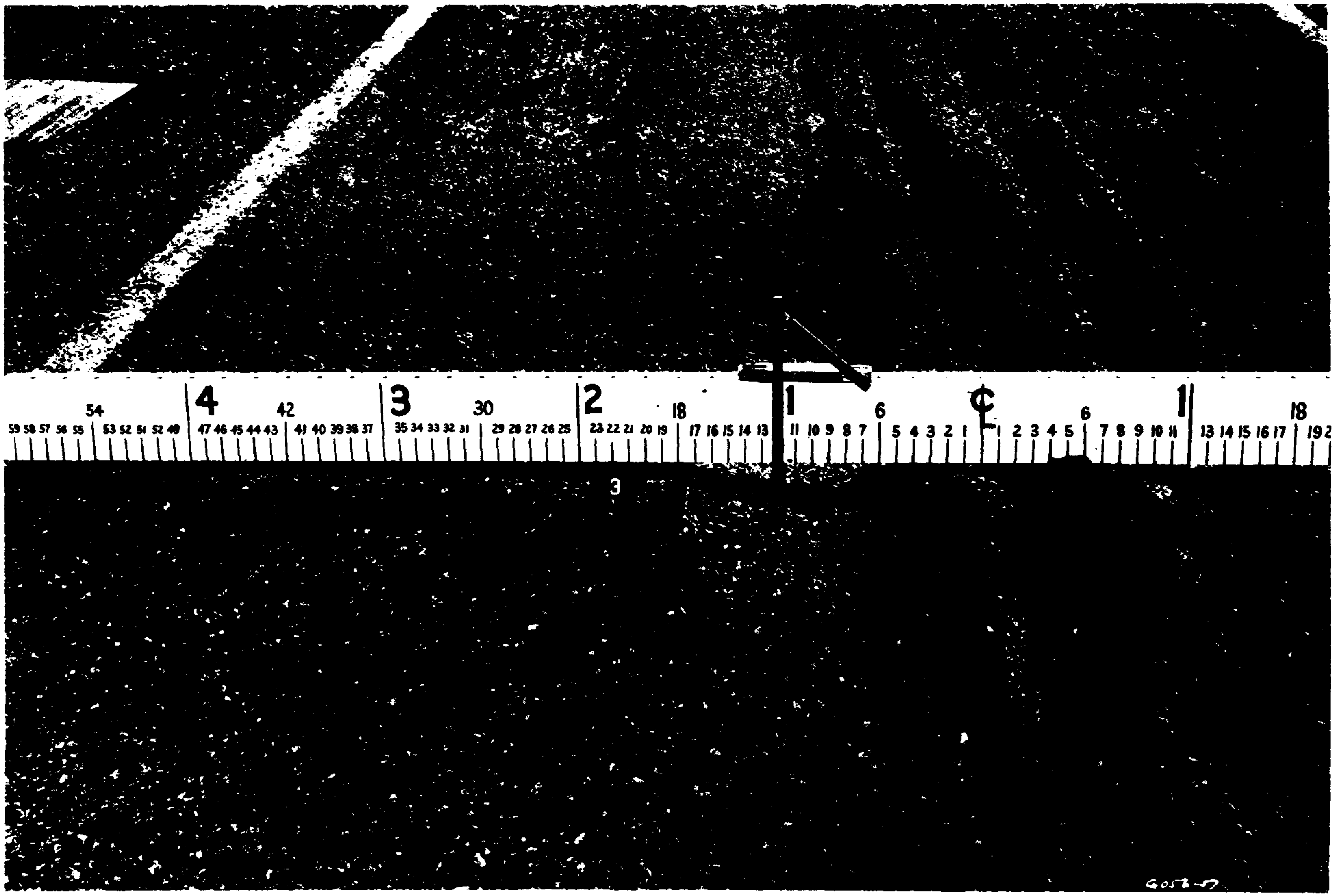


PHOTO C-14. Failure of DBST (Item 3) Under Distributed Traffic (48 Passes)

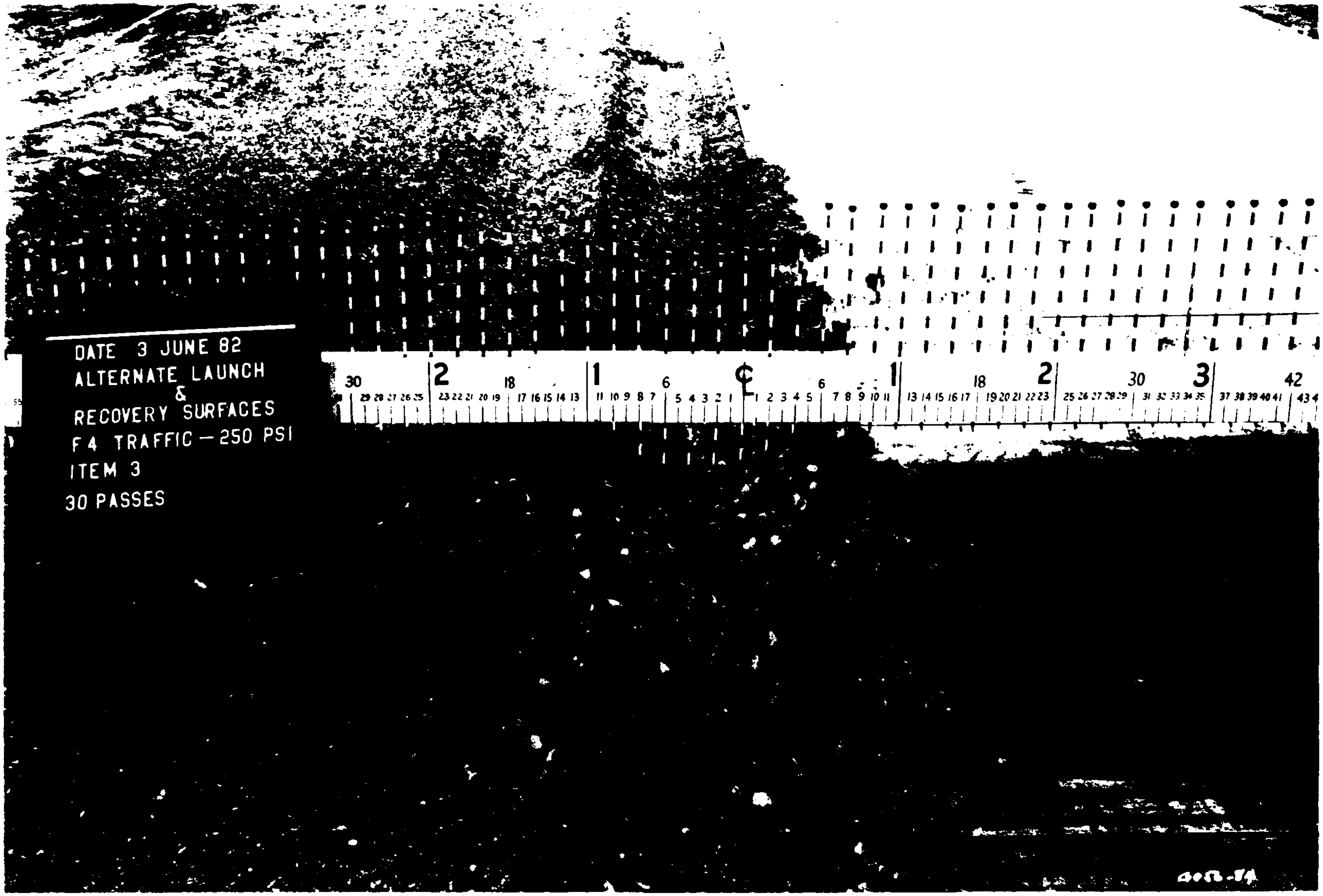


PHOTO C-15. Failure of DBST (Item 3) Under Channelized Traffic (30 Passes)

PHOTO C-16. Failure of DBST (Item 3) After One Locked-Wheel Skid Test

PHOTO C-17. Typical View of Test Trench After Removal of Pavement Surface

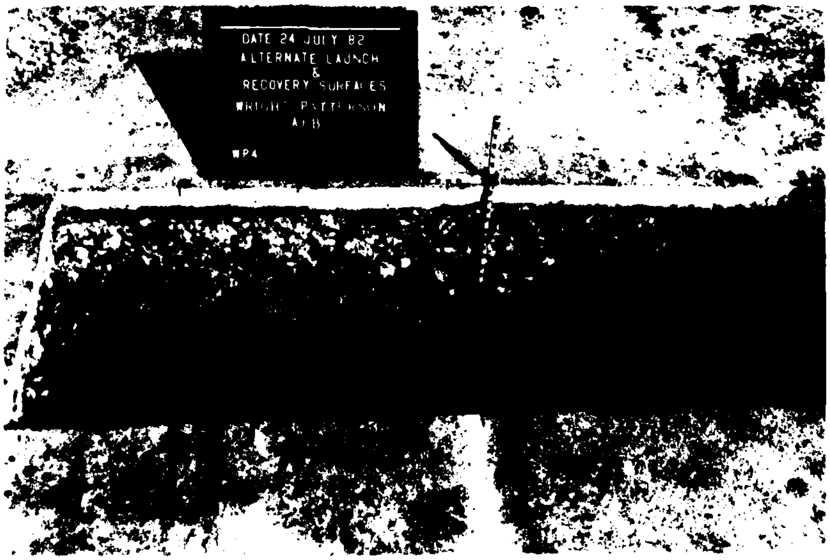


PHOTO C-18. View Showing Test Trench After Removal of the Pavement Surface and the Base Material

PHOTO C-19. General View of Test Area (WP-1) Before Traffic

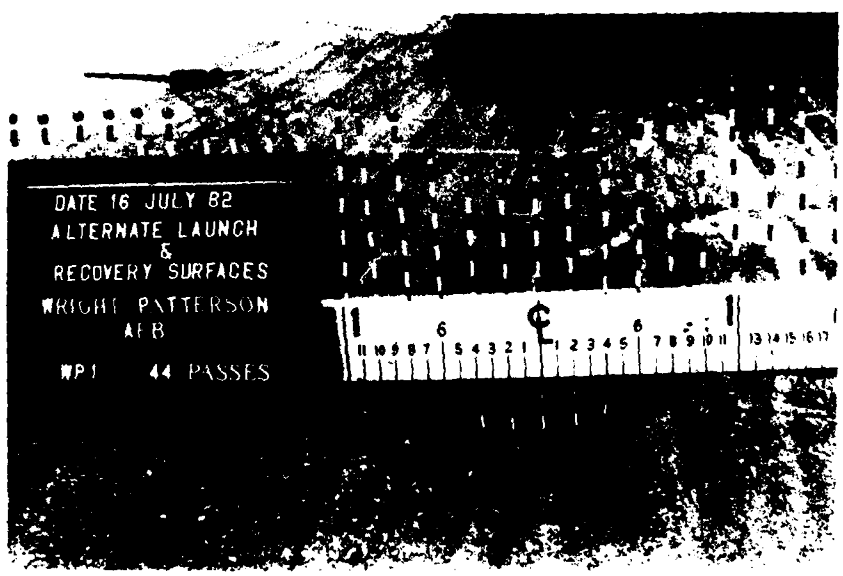


PHOTO C-20. Closeup of Measurement Showing Maximum Deformation at Failure

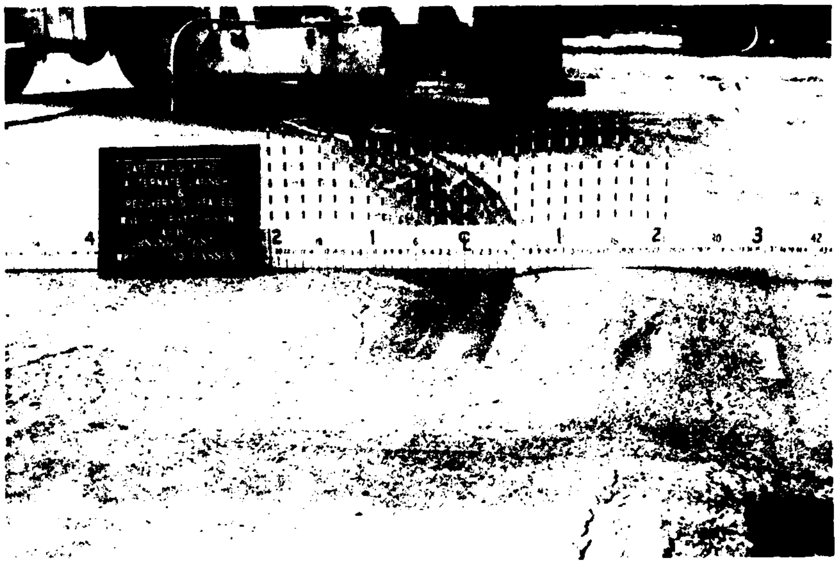


PHOTO C-21. View Showing 2-inch Rut Depth After 10 Passes of the Turning Test

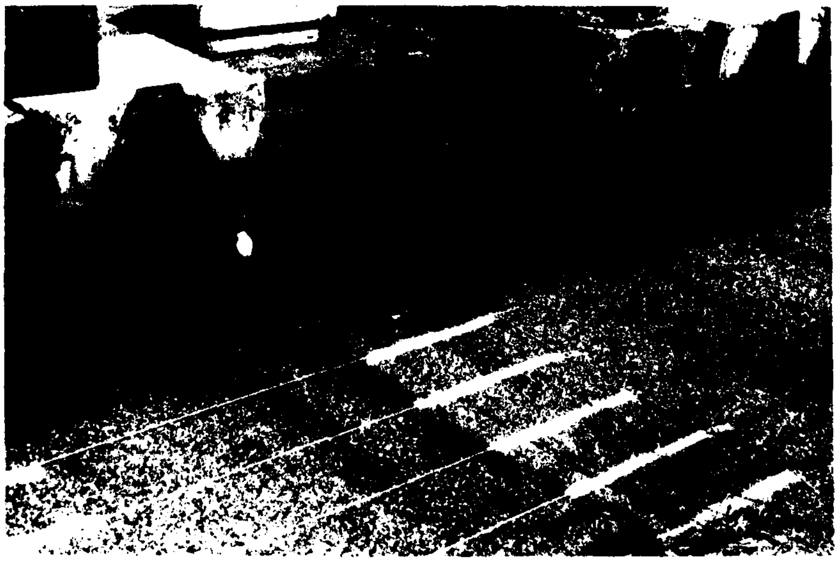


PHOTO C-22. View of F-4 Load Cart Wheel in Skid Rut

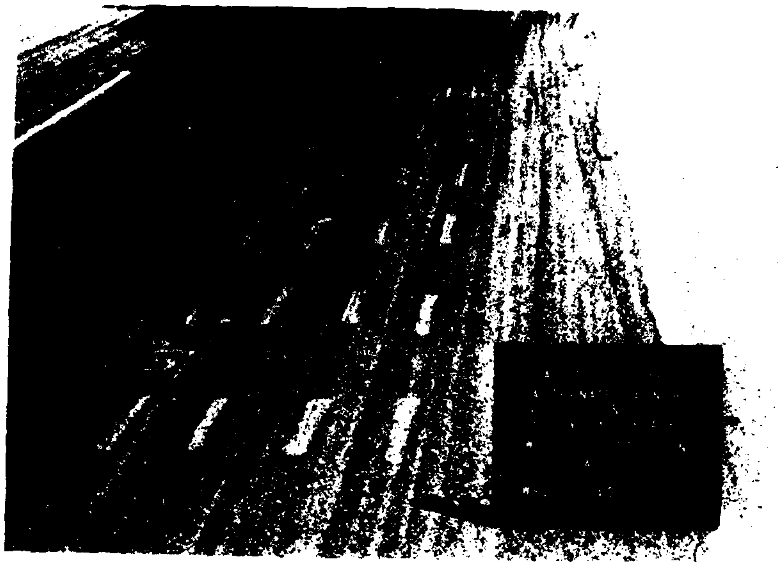


PHOTO C-23. View Showing Good Condition of Traffic Area After 48 Passes

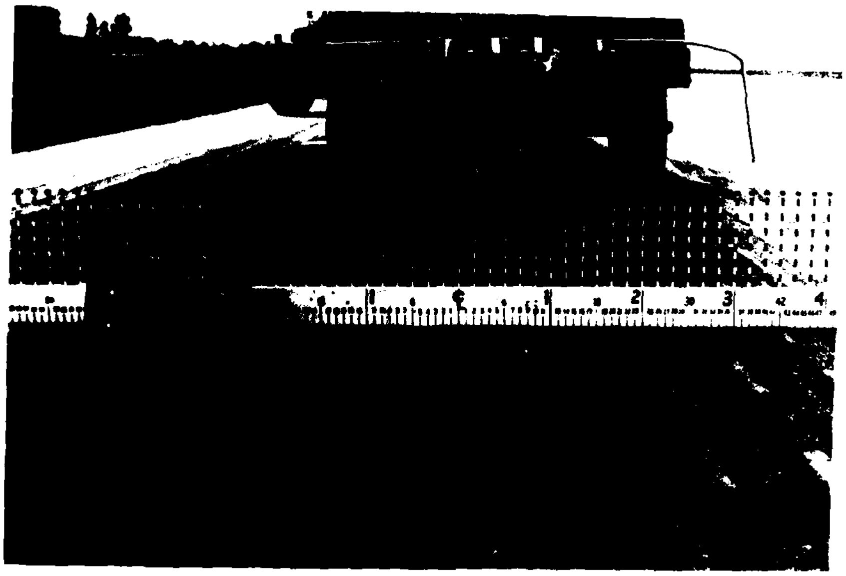


PHOTO C-24. WP-2 Test Feature After 240 Passes

PHOTO C-25. View Showing Alligator Cracking After 481 Passes

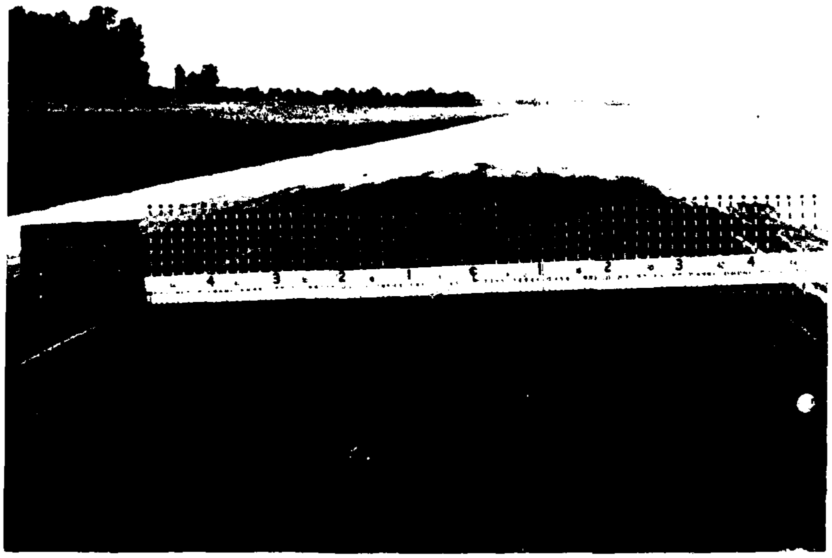


PHOTO C-26. Failure of Test Features WP-2, Rut Depth in Excess of 3 Inches

AD-A145 800 DESIGN OF ALTERNATE LAUNCH AND RECOVERY SURFACES FOR ENVIRONMENTAL EFFECTS(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS GEOTE.. 3/3

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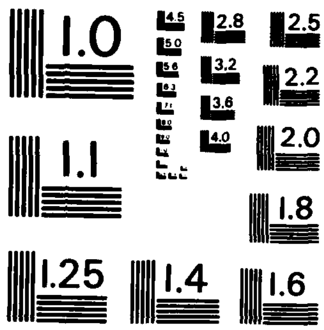

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

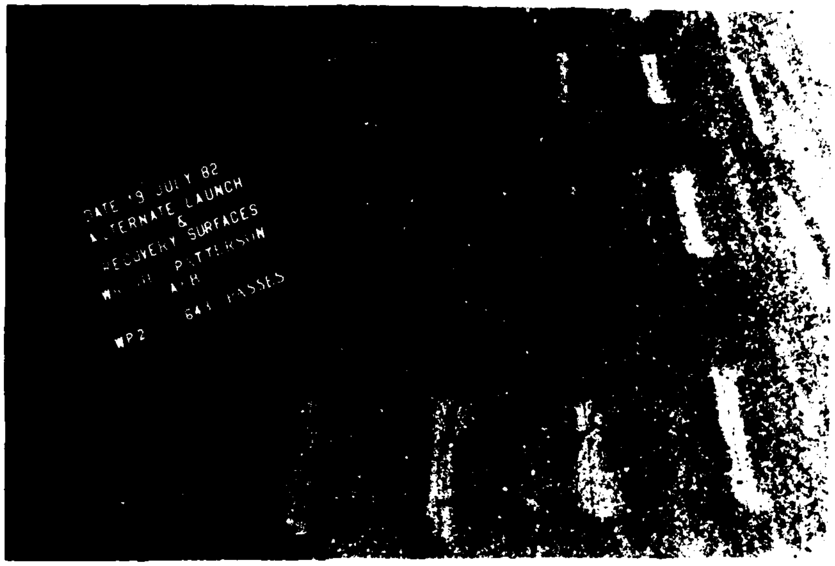

PHOTO C-27. Closeup View of Failed Condition, WP-2

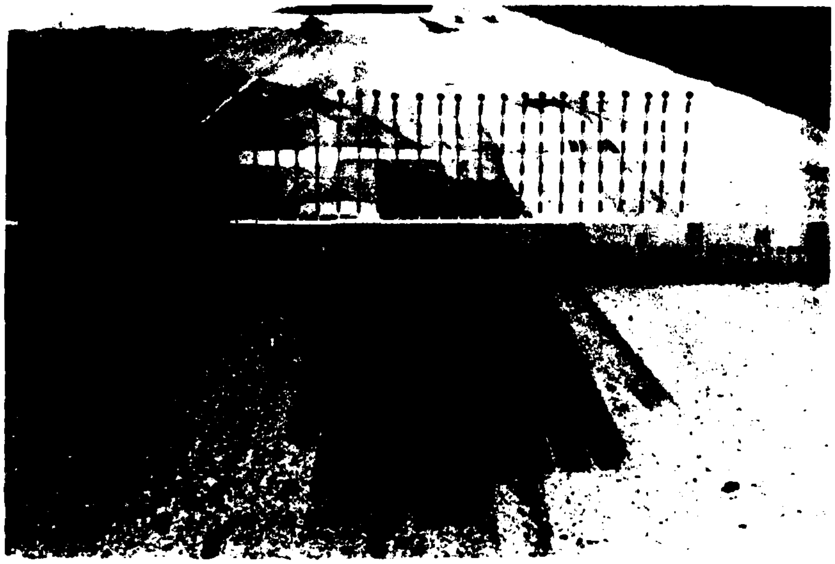

PHOTO C-28. Straightedge Measurement (1/4 Inch) After 6 Skid Tests

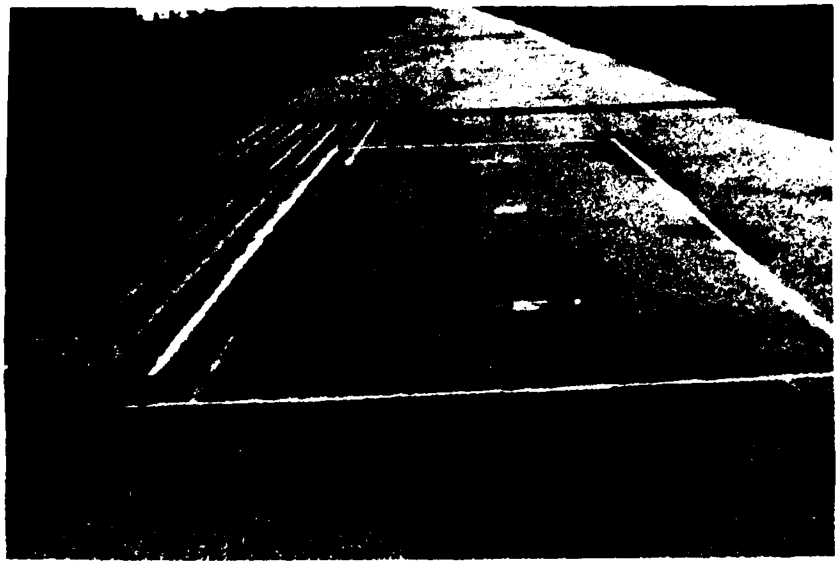

PHOTO C-29. General View of Test Area WP-3 Prior to Start of Traffic

PHOTO C-30. Failed Condition of WP-3 After 90 Passes

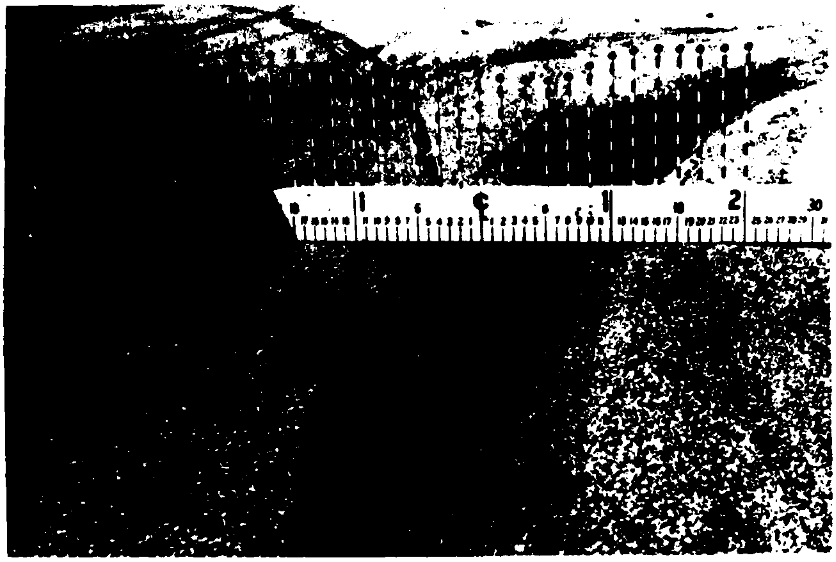

PHOTO C-31. Deformation Measurements After 28 Passes of Turning Test

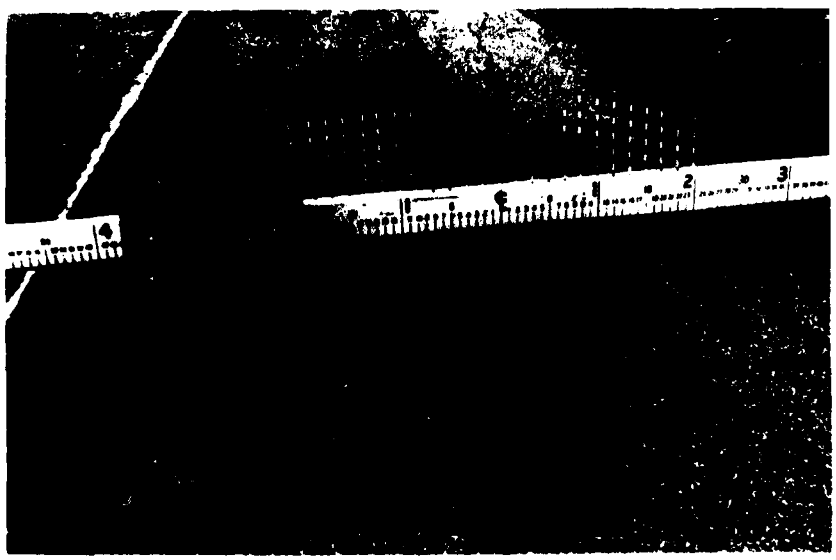

PHOTO C-32. Failure After Six Locked-Wheel Skid Tests; the Rut Depth Measured 10 Inches

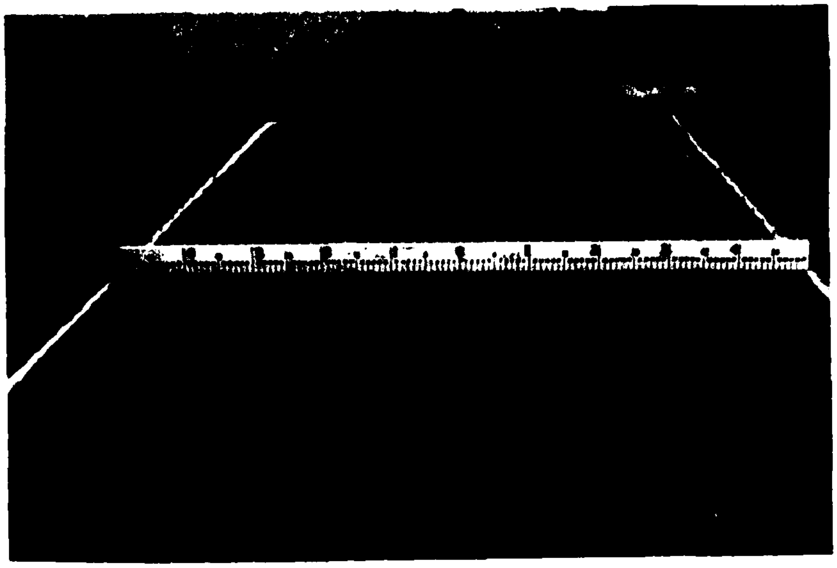

PHOTO C-33. View Showing the Very Good Pavement Condition of WP-4 After 48 Passes

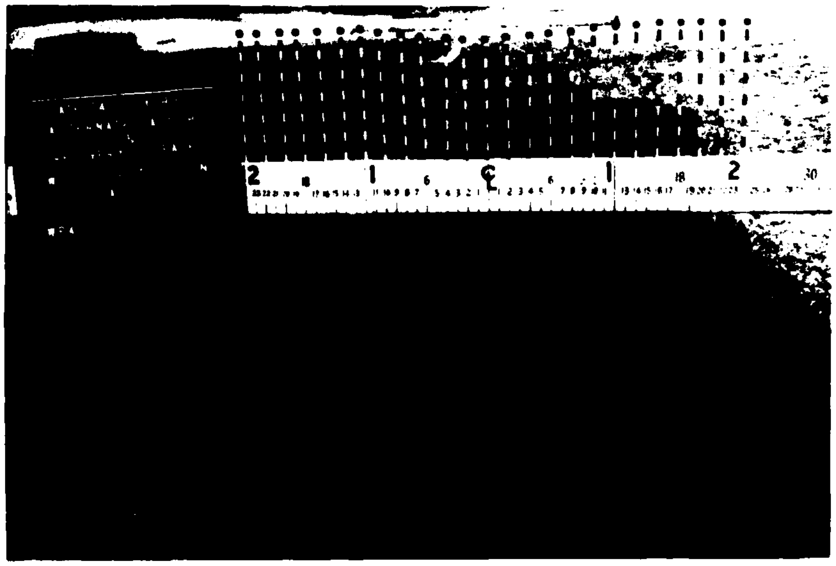

PHOTO C-34. General View of Turning Test Area After 75 Passes

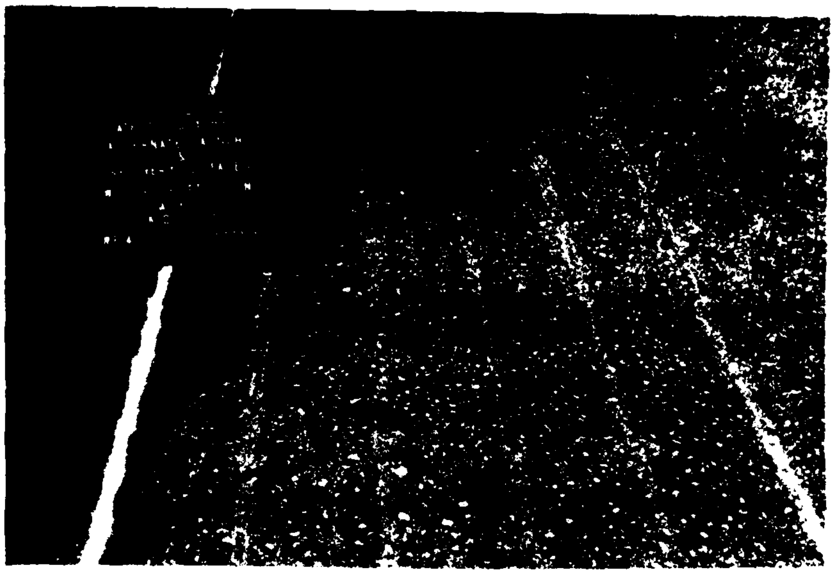

PHOTO C-35. Locked-Wheel Skid Test Area Before Testing

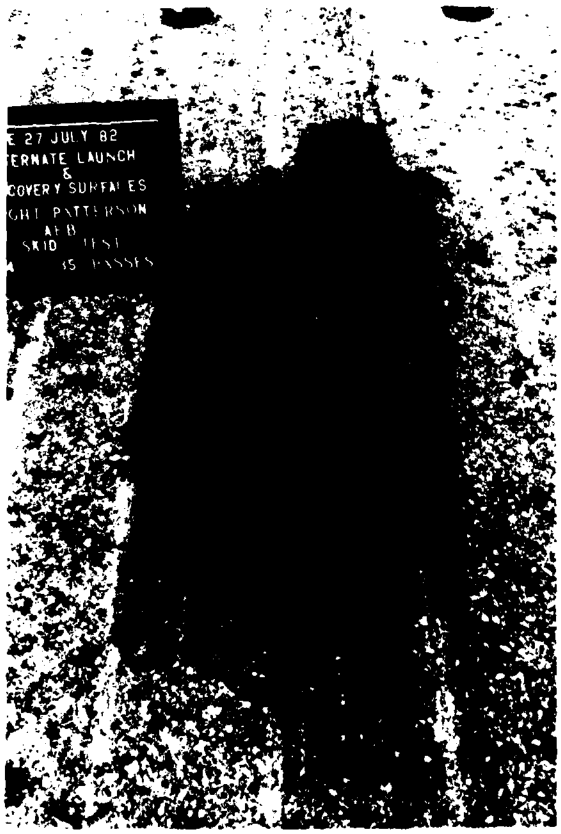

PHOTO C-36. Same Area After Failure

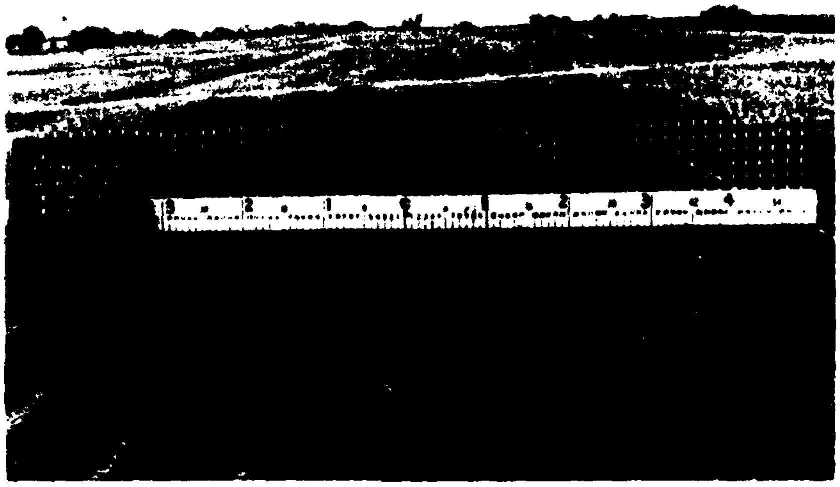

PHOTO C-37. Breaking Up of DBST After 100 Passes on Test Feature W-1

PHOTO C-38. Failed Condition of the W-1 Feature; Rut Depth in Excess of 3 Inches

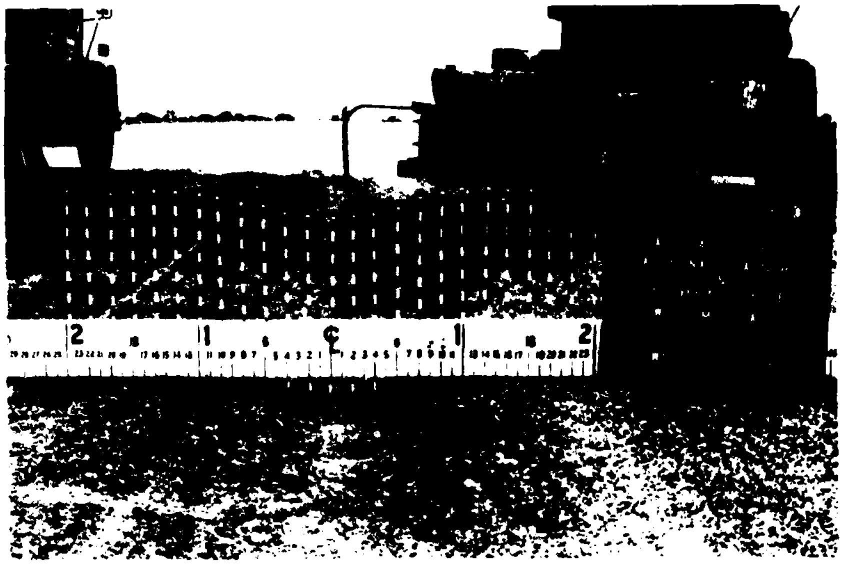

PHOTO C-39. View of Turning Test Area After Completing 75 Passes

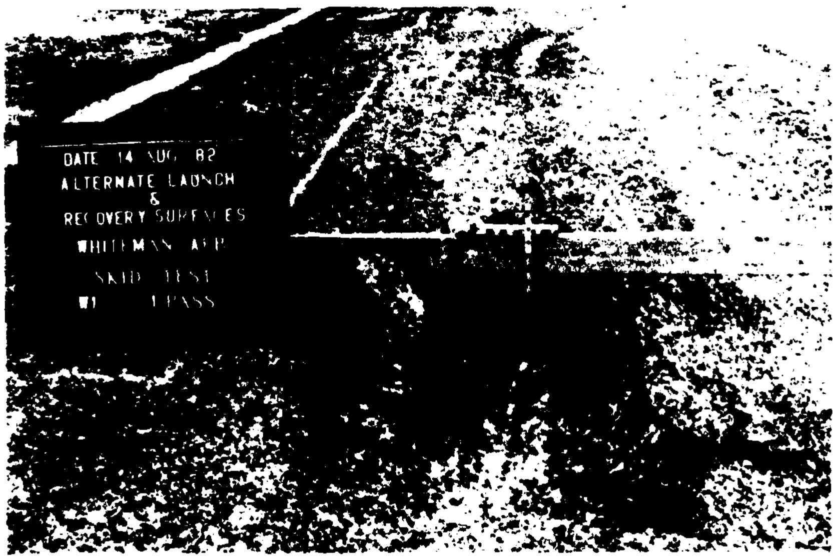

PHOTO C-40. 4 1/2-inch Rut Depth Measured After One Skid of the Load Cart

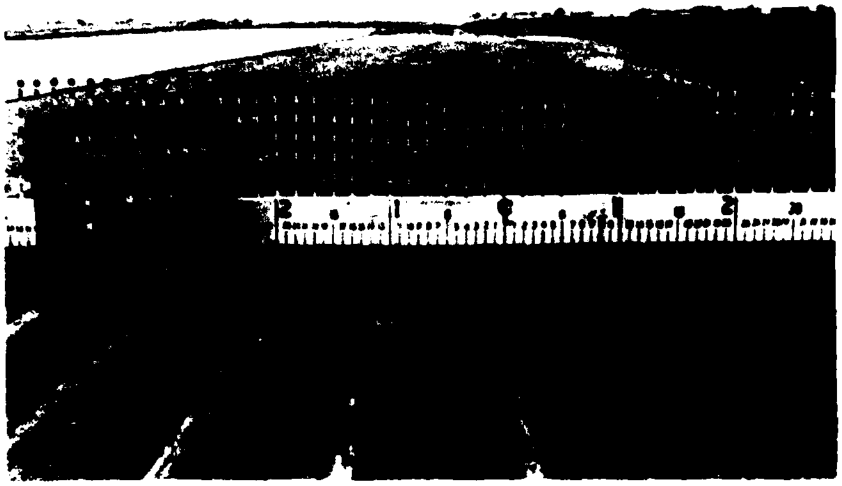

PHOTO C-41. View of Traffic Area, W-2, After 50 Passes

PHOTO C-42. View After 100 Passes Showing Cracking in Pavement Surface and a Rut Depth of 2 3/8 Inches

PHOTO C-43. Failed Condition After 132 Passes of the Load Cart

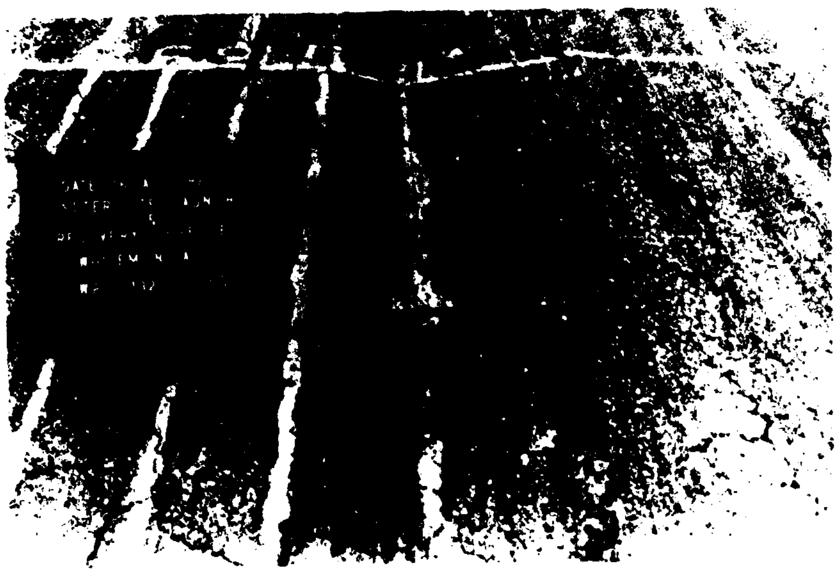

PHOTO C-44. Closeup of Failed Area Showing Loose Material that Spalled from the Cracks

PHOTO C-45. View of Turning Test Area Before Traffic

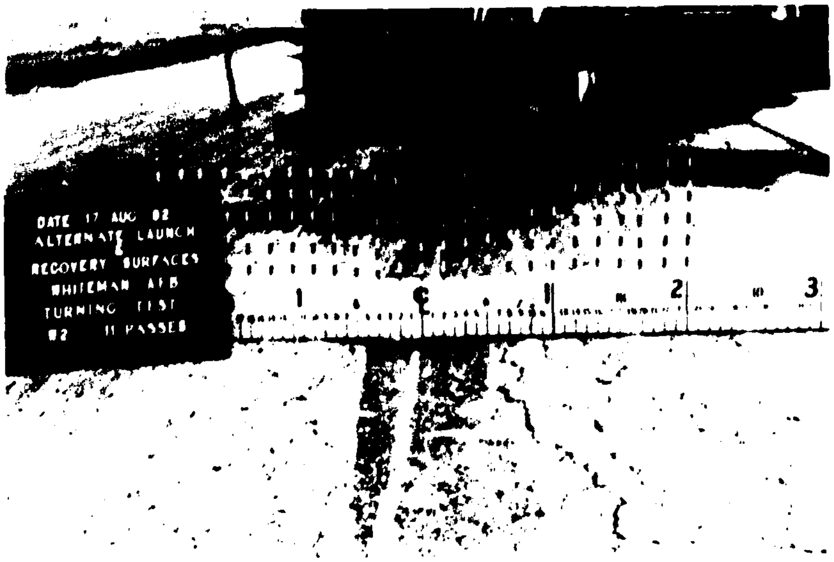

PHOTO C-46. View of 1 1/4-inch Rut Depth After Completing 11 Turns

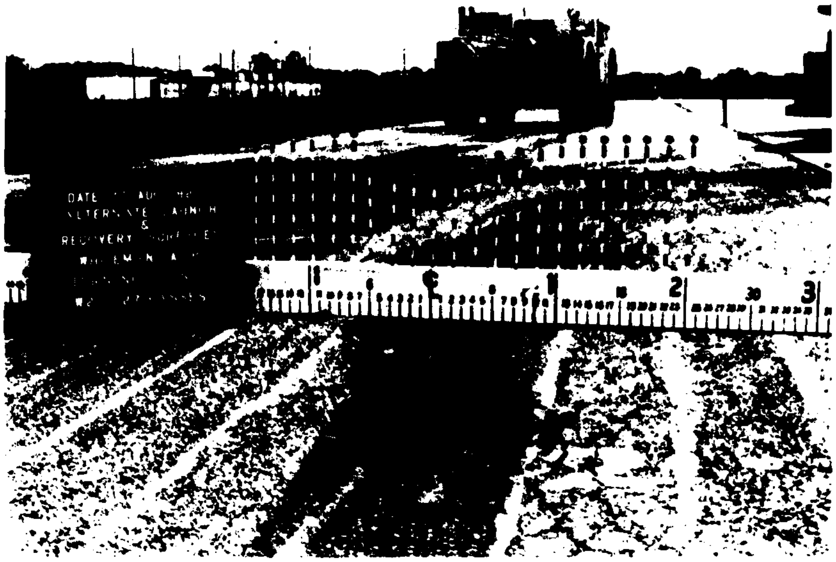

PHOTO C-47. View of Turning Test Area After Failure

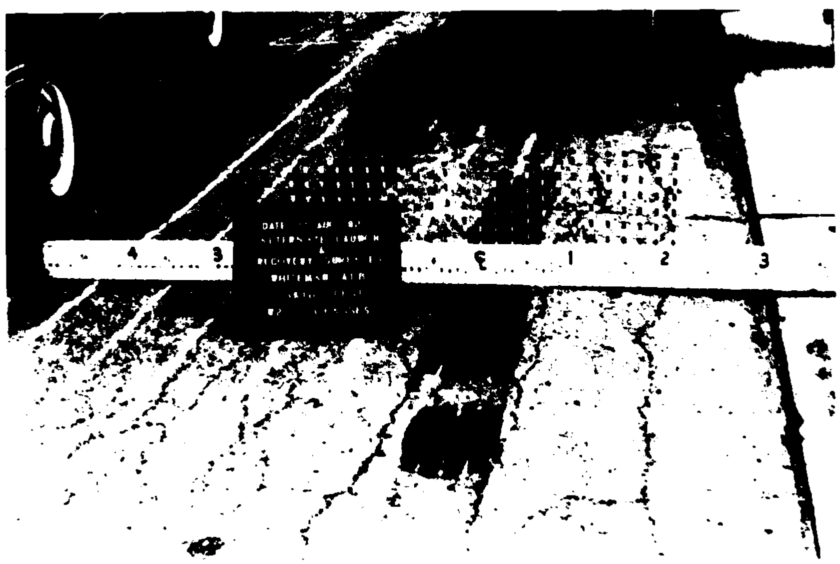

PHOTO C-48. Pavement Condition After Three Skid Tests

PHOTO C-49. General View of Pavement Condition in Apron Area Where Test Area W-3 was Located

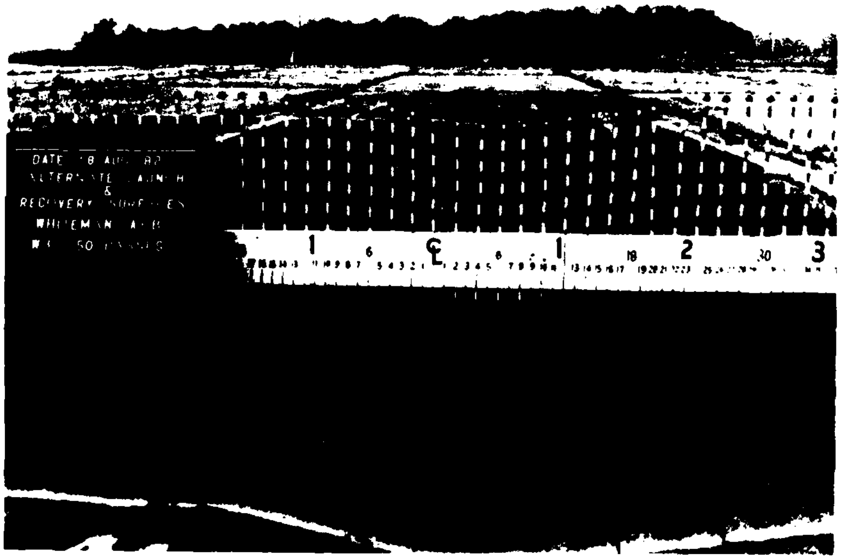

PHOTO C-50. Test Area W-3 After 50 Passes

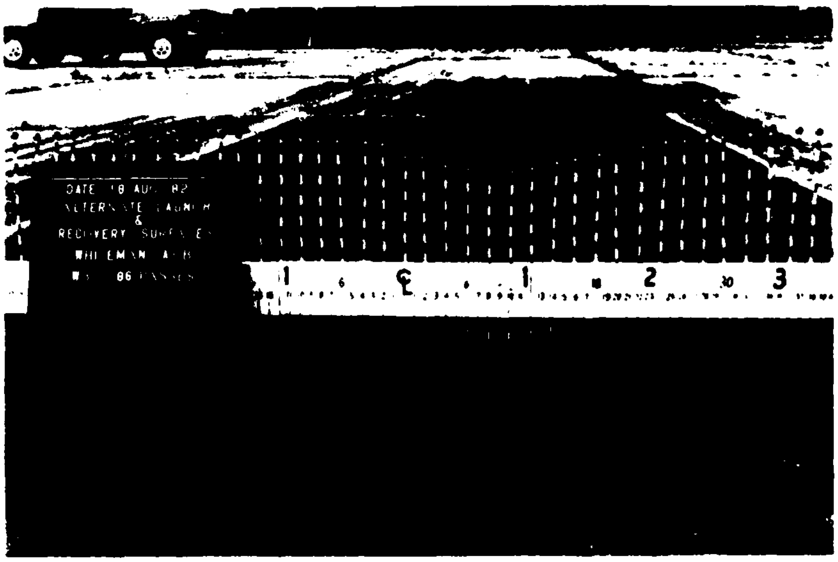

PHOTO C-51. Pavement Failure, Extensive Cracking and Rut Depth of 3 1/2 Inches

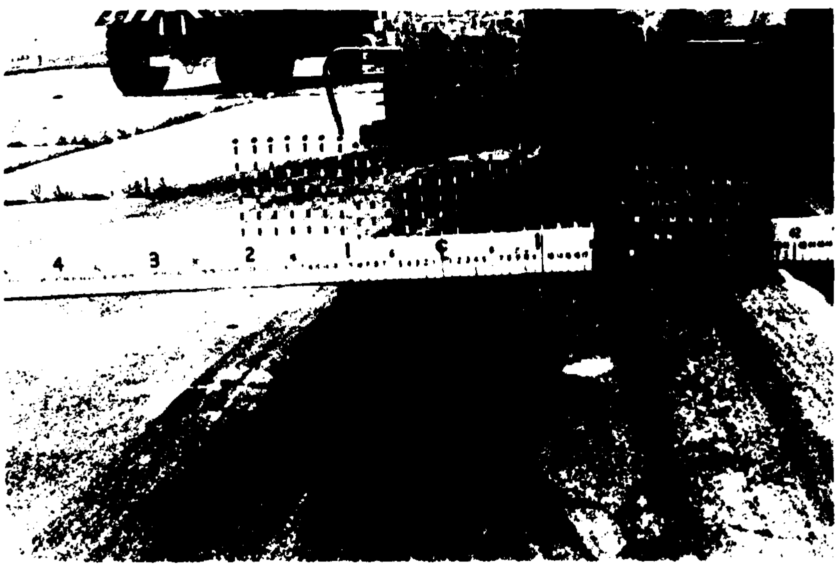

PHOTO C-52. Pavement Failure After 27 Turning Passes

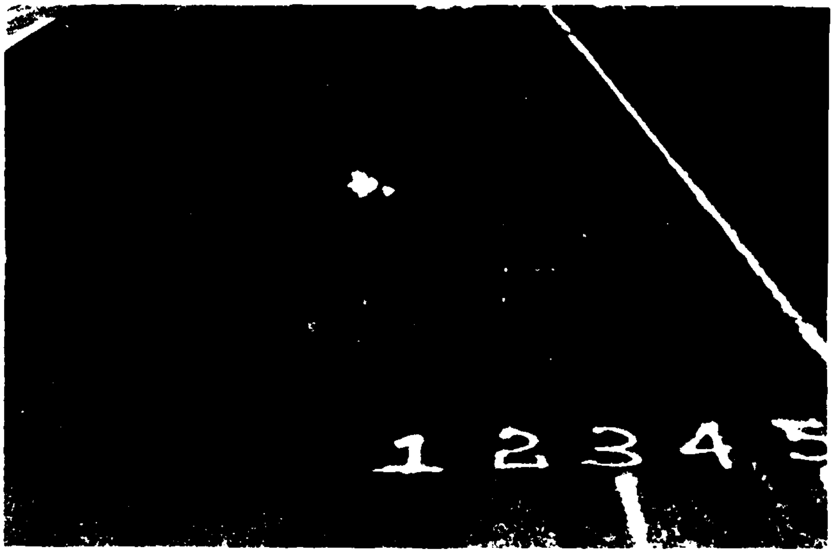

PHOTO C-53. W-3 Skid Test Pavement Area Before Traffic

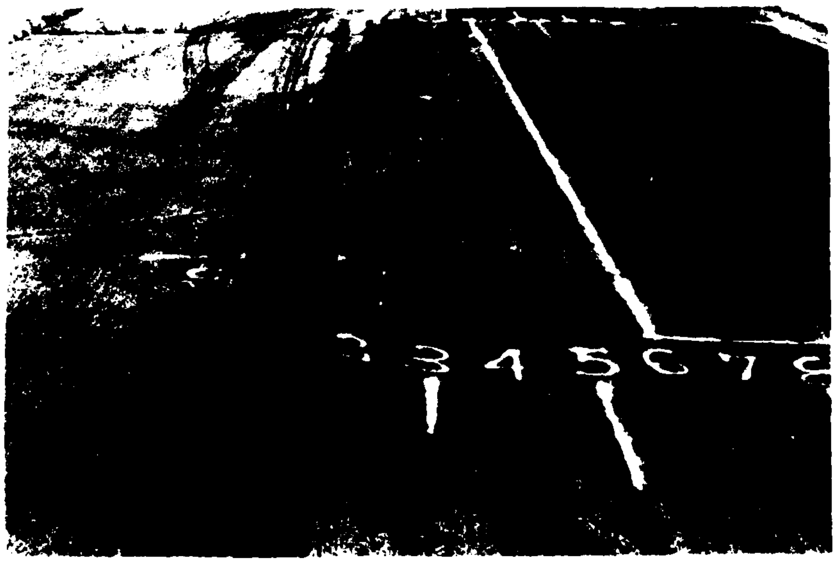

PHOTO C-54. Same Area After Failure

PHOTO C-55. Example of Surface Drainage Problem, Hahn ALRS

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